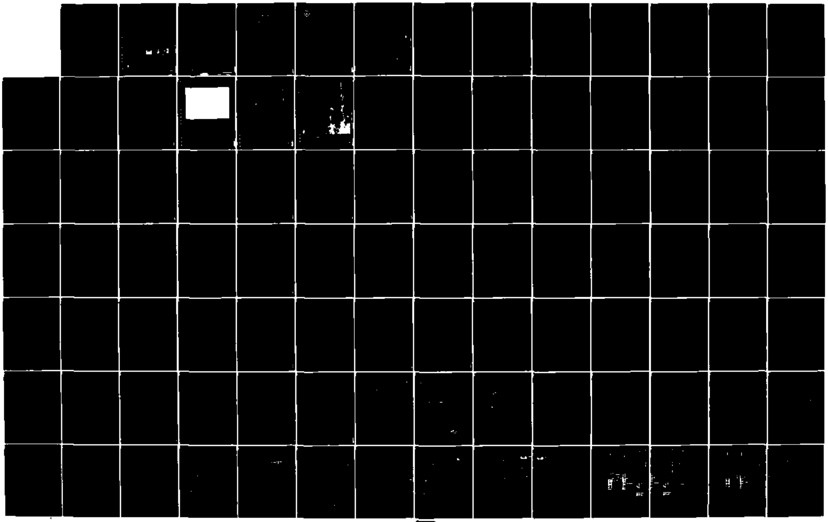
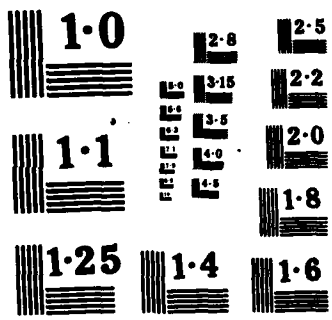
AD-A157 626 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS 1/2
NORTON BROOK DAM (VT..(U) CORPS OF ENGINEERS WALTHAM MA
NEW ENGLAND DIV JAN 80

UNCLASSIFIED F/G 13/13 NL




1·0
2·8
2·5
3·15
2·2
3·5
2·0
4·0
1·1
4·5
1·8
1·25
1·4
1·6

(1)

AD-A157 626

RICHELIEU RIVER BASIN
TOWN OF BRISTOL
ADDISON COUNTY, VERMONT

NORTON BROOK DAM
VT 00102

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MA 02154

JANUARY 1980

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
VT 00102

2. GOVT ACCESSION NO.
AD-A157 626

3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle)
Norton Broek Dam

NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS

5. TYPE OF REPORT & PERIOD COVERED
INSPECTION REPORT

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

8. CONTRACT OR GRANT NUMBER(s)

9. PERFORMING ORGANIZATION NAME AND ADDRESS

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS

11. CONTROLLING OFFICE NAME AND ADDRESS
DEPT. OF THE ARMY, CORPS OF ENGINEERS
NEW ENGLAND DIVISION, NEDED
424 TRAPELO ROAD, WALTHAM, MA. 02254

12. REPORT DATE
January 1980

13. NUMBER OF PAGES
107

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)

15. SECURITY CLASS. (of this report)
UNCLASSIFIED

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)
APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES
Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)
DAMS, INSPECTION, DAM SAFETY,

Richelieu River Basin
Norton Brook
Town of Bristol
Addison County, VT

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

he

The dam is an earth embankment consisting of a dam section and an immediately adjacent dike section. The dam is about 354 ft. long and about 34 ft. high. The dam is in poor condition, primarily because of the presence of sinkhole-like features upstream from the core wall of both the dam and dike. It is small in size with a significant hazard potential. There are various remedial measures and recommendations which ~~must~~ should be implemented by the owner.

DISCLAIMER NOTICE

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF
NEDED

MAY 30 1980

Honorable Richard A. Snelling
Governor of the State of Vermont
State Capitol
Montpelier, Vermont 05602

Dear Governor Snelling:

Inclosed is a copy of the Norton Brook Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Water Resources, the cooperating agency for the State of Vermont. In addition, a copy of the report has also been furnished the owner, the city of Vergennes, Vergennes, Vermont.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Water Resources for your cooperation in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

RICHELIEU RIVER BASIN
TOWN OF BRISTOL
ADDISON COUNTY, VERMONT

NORTON BROOK DAM
VT 00102

Accession For	
NTIS GRA&I	☒
DTIC TAB	☐
Unannounced	☐
Justification	
By	
Distribution/	
Availability Codes	
Dist	Avail and/or Special
A/1	23

DTIC COPY INSPECTED 3

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

GORDON E. AINSWORTH & ASSOCIATES, INC.
Engineers, Surveyors and Planners
20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

PROJECT NO. 21.06.79103

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.:	VT 00102
Name of Dam:	Norton Brook Dam
Town:	Bristol
County and State:	Addison County, Vermont
Stream:	Norton Brook
Date of Inspection:	24 October 1979

BRIEF ASSESSMENT

1. Project Description

Norton Brook Dam is an earth embankment consisting of a dam section and an immediately adjacent dike section. The two sections are barely separated by a narrow natural rock outcrop abutment. The dam and dike combined are about 597 feet long.

The dam section is 354 feet long and about 34 feet high. The upstream slope is about 2.5H:1V and the downstream slope is about 2H:1V. Top width varies from 4 to 8 feet. The abutments of the dam are bedrock. The dam is set on a soil foundation with a concrete core wall that partially penetrates the foundation soils. Approximately one-half of the length of the core wall has a sheet pile wall to bedrock beneath the core.

The dike section, to the left of the dam, is about 243 feet long and 17 feet high. The upstream slope is about 2H:1V and the downstream slope is about 1.5H:1V. Top width is about 4 feet. All other features of the dike are similar to the dam, except that no sheet piling was used under the core wall.

Normal pool elevation of 4 feet below the top of the dam and dike is maintained by a single drop inlet spillway located about at the midpoint of the dam section. This is the only spillway for the dam.

2. Significant Findings and Assessment

From a geotechnical standpoint, the dam is in POOR condition, primarily because of the presence of sinkhole-like features upstream from the core wall of both the dam and dike. Also, numerous large trees and brush cover all surfaces, a beaver pond obscures observation of any potential seeps downstream of the central part of the dam, and substantial settlement of the crest relative to the core wall has occurred.

3. Hydraulic and Hydrologic Findings

The dam has adequate spillway capacity, because the test flood does not overtop the dam. In accordance with recommended guidelines established by the Corps of Engineers, the dam is classified as SMALL in size and as having a SIGNIFICANT hazard potential. Accordingly, a TEST FLOOD equal to ONE-HALF PMF (probable maximum flood) was judged as appropriate within the recommended range of the 100-year flood to one-half PMF. The test flood does not overtop the dam, but results in a minimum freeboard of about 2.2 feet. Peak inflow for the test flood is 330 cfs. Peak outflow is 170 cfs. Total project discharge capacity at the top of the dam is due only to the drop inlet spillway (outlet works assumed closed) and is equal to 540 cfs, or 318% of the test-flood peak outflow.

4. Recommended Action

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the following recommendations:

a. All trees and brush should be removed from the surfaces to a distance of 20 feet downstream from the toeline and the surfaces kept mowed. The beaver pond downstream should be removed.

b. A registered engineer qualified in the design of dams should be engaged to investigate the sinkhole-like feature upstream of the dike and the depression upstream of the dam, to recommend how to fill tree rootholes, to recommend repairs to the outlet structure training walls, to inspect the dam after the surfaces have been cleared and the beaver pond downstream has been removed, and to make recommendations for monitoring the seeps.

Additional recommendations and remedial measures that should be implemented by the Owner WITHIN ONE YEAR after their receipt of this Phase I Inspection Report are described in Section 7.

GORDON E. AINSWORTH & ASSOCIATES, INC.

Kenneth J. Male, P.E.

LICENSED PROFESSIONAL ENGINEER
KENNETH J. MALE
25004
UNIVERSITY OF THE STATE OF NEW YORK
& LAND SURVEYOR

This Phase I Inspection Report on Norton Brook Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Aramast Mahtesian

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Richard J. DiBuono

RICHARD DIBUONO, CHAIRMAN
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation: however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external con-

ditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does <u>not</u> include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

NORTON BROOK DAM

PHASE I INSPECTION REPORT

TABLE OF CONTENTS

	Page
Letter of Transmittal	-
Brief Assessment	1
Review Board Page	-
Preface	i
Table of Contents	iii
Overview Photo	vii
Location Map	viii
Vicinity Map	ix
Section	
1 - PROJECT INFORMATION	
1.1 General	
a. Authority	1-1
b. Purpose of Inspection	1-1
1.2 Description of Project	
a. Location	1-1
b. Description of Dam and Appurtenances	
1) Main Dam	1-2
2) Diversion Dam	1-3
c. Size Classification	1-4
d. Hazard Classification	1-4
e. Ownership	1-4
f. Operator	1-4
g. Purpose of Dam	1-5
h. Design and Construction History	1-5
i. Normal Operation Procedures	1-5

d. Reservoir (length in feet)

1) Maximum Pool Length - Spillway Crest 1,040 ±
- Top of Dam 1,060 ±

2) Shoreline - Spillway Crest 3,570 ±
- Top of Dam 3,640 ±

e. Storage (acre-feet)

1) Spillway Crest 170

2) Top of Dam 233

3) Test Flood Pool 196

f. Reservoir Surface (acres)

1) Spillway Crest 14.7

2) Top of Dam 17.0

3) Test Flood Pool 15.7

g. Dam			Dike
1)	Type -	Earth	Earth
2)	Length -	354 feet	243 feet
3)	Height Hydraulic Height - 34 feet Structural Height - 36 feet		 17 feet 22 feet
4)	Top Width - Varies 4 to 8 feet		4 feet
5)	Side Slopes Upstream - Downstream -	 2.5H:1V 2H:1V	 2H:1V 1.5H:1V
6)	Slope Protection Upstream - Downstream -	 Rock Riprap Vegetation	 Rock Riprap Vegetation
7)	Approximate Volume - 20,000 cubic yards		2,500 cu.yds.
8)	Zoning -	None known	None known

3) Computed Discharge - W.S. at Test Flood Elevation

Outlet Works	None
Ungated Drop Inlet Spillway	170
Gated Spillway	N/A
Over Dam	N/A
Total Project	Same as Spillway

c. Elevation (feet - NGVD)

A note on the original design/construction drawings (Appendix B2-1) indicates that the elevations on the drawings are "referred to USGS Datum, mean sea level at Sandy Hook EL 0.00". Comparing the drawing elevation shown at a point on Plank Road where it is crossed by the Little Otter Creek (EL 310) and at normal reservoir level (EL 410) with comparable points on the USGS map (less than EL 280 and EL 381 respectively per Appendix D-1), there is a 29-foot difference. Therefore, all elevations used in this report are 29 feet less than those on the original design/construction drawings in Appendix B and are in approximate feet above mean sea level NGVD (National Geodetic Vertical Datum of 1929).

1)	Natural Streambed at Toe of Dam - Upstream	353 ±
	-Downstream	351 ±
2)	Lowest Foundation Surface (core wall bottom)	349
3)	Core Wall - Bottom (lowest point)	349
	- Top	384
4)	Bottom of Cutoff (lowest point-cutoff only exists under portion of dam)	319 ±
5)	Maximum Tailwater	Unknown
6)	Recreation Pool	N/A
7)	Flood Control Pool	N/A
8)	Normal Pool	381
9)	Spillway Crest (ungated drop inlet)	381
10)	Design Surcharge	Unknown
11)	Top of Dam and Dike	385
12)	Test Flood Surcharge	382.8

The drop inlet spillway is uncontrolled and wide open. The water main intakes are partially open supplying water to the 17 families still tapped into the otherwise shut-down transmission main. The low level drain is closed. The 15-inch diameter pipe supplying inflow from Rivers Brook Diversion Dam appears to be valved off at the diversion dam, but its exact status is not clear.

Refer to Section 4 of this report for a complete discussion of operation and maintenance procedures.

1.3 Pertinent Data

a. Drainage Area

1) Location - West central Vermont in northwestern foothills of Green Mountain National Forest.

2) River Basin - Tributary to Norton Brook, then to Rivers Brook, to Little Otter Creek, to Lake Champlain, to Richelieu River.

3) Shape - Roughly rectangular, about 2,000 feet by 2,400 feet.

4) Area - 0.155 square miles, or 99.4 acres.

5) Topography - Fairly steep wooded slopes averaging 25% slope. Elevations vary from EL 381 to EL 847.

6) Other - Additional inflow via 15-inch pipeline from Rivers Brook Diversion Dam from its 0.976-square mile drainage area.

b. Discharge at Dam Site (cfs)

1) Maximum Known Flood	Unknown
2) Computed Capacity - W.S. at Top of Dam	
Outlet Works	
Spillway Outlet Conduit	540
Low Level Drain (normally closed)	Not Estimated
Water Supply Intake (in minimal use)	Unknown
Ungated Drop Inlet Spillway	540
Gated Spillway	None
Total Project	540

g. Purpose of Dam

The dam was originally constructed to provide, and did provide until 1972, an active water supply for the City of Vergennes. Since 1972, the City has stopped using the reservoir as its water supply and has drawn water from Lake Champlain. There are 17 families who live along the route of the transmission main to the City who are still tapped into the main and who use raw water from the reservoir as their water supply. The City would like to shut the transmission main down entirely, but the 17 families who still use the water have a suit against the City trying to force the City to continue to provide them with water.

The City would like to sell the dam and reservoir, and/ or see it developed for recreation without the City's involvement. Presently, a local fish and game club, who have a club building and firing range in a field about 1,700 feet downstream of the dam (see Photo C-13A), have a long-term lease on the approximate 10-acre parcel of ground occupied by the firing range and club building. However, they have no lease or other rights on the dam and reservoir.

h. Design and Construction History

The dam was constructed in 1935 for the City of Vergennes. The designer was Barker and Wheeler Engineers, 36 State Street, Albany, New York, who are no longer in business and the location of whose files is unknown. The construction contractor was W. G. Fritz Company, 69 Main Street, West Orange, New Jersey. The business status of this firm and the location of its files are unknown.

On June 21, 1942, part of the dike washed out, exposing and undercutting the core wall. Repair work was done by a "... Mr. Overacked of Burlington...". The whereabouts of this gentleman and of any records he may have of his repair work are unknown. From one photo showing the repair work underway (Appendix B3-11), it would appear that the core wall was extended deeper during the repair.

No other construction, modification, or major repair are known to have occurred. Refer to Section 2 of this report for a complete discussion of the design, construction and operation history, with selected plans and other engineering data included in Appendix B.

i. Normal Operation Procedures

Since 1972 when the City of Vergennes stopped using the dam and reservoir as an active water supply, the City has essentially abandoned the operation and maintenance of the dam and reservoir. Consequently, there are no current operation and maintenance procedures.

c. Size Classification

In accordance with recommended guidelines (Reference 1), Norton Brook Dam is classified as SMALL in size because its maximum height is 34 feet (within the 25 to 40-foot range), and also because its maximum storage is 233 acre-feet (within the 50 to 1000-acre-foot range).

d. Hazard Classification

In accordance with recommended guidelines (References 1 & 18) involving urban development and economic loss, Norton Brook Dam is classified as having a SIGNIFICANT hazard potential. The dam is located in a predominantly rural or agricultural area where failure could damage "no more than a small number of habitable structures" (approximately two), and do "minimal to appreciable damage" to portions of a light-duty highway (Plank Road) and to some agricultural land (along the Little Otter Creek). There appears to be potential for future development in the hazard area. Also, a dam failure would disrupt the water supply for the 17 families along the water transmission main who still use water from the reservoir. The dam failure analysis is developed in Section 5.5 of this report.

e. Ownership

Since its construction, the dam and reservoir have been and are still owned by:

City of Vergennes
P.O. Box 169
Vergennes, Vermont 05491

Attention: Kenneth C. Thiess, City Manager
(802) 877-3637

The City also owns most, if not all, of the watershed, including Rivers Brook Diversion Dam.

f. Operator

Day-to-day operation of the dam is the responsibility of:

Carroll O'Connor, Supervisor of Public Works
(802) 877-3637
(Same address as Owner.)

concrete spillway outlet conduit about 101 feet long through the dam and core wall, discharging to Norton Brook, and having a horseshoe-shaped cross-section 4 feet wide by 6 feet high.

On the upstream side of and integral with the spillway structure there is a concrete control tower and intake structure. Three valved intakes at different levels feed into a single 8-inch cast iron water supply pipe in the bottom of the valve chamber under the control tower. The 8-inch pipe continues through the valve chamber wall into the spillway outlet conduit, where it is supported about half way up the conduit wall and wrapped with insulation as it runs inside the conduit. Just before the outlet of the conduit, the 8-inch pipe goes through the conduit wall into the ground, and then increases to a 10-inch transmission main, which continues cross-country about 6 miles to the City of Vergennes.

Under the bottom of the intake structure, through the bottom of the valve chamber, and discharging into the spillway outlet conduit, there is a valved 14-inch diameter low level outlet pipe.

2) <u>Diversion Dam</u>

In addition to receiving flow from a small natural drainage area, Norton Brook Dam receives inflow (estimated at 7 cfs) from a small diversion dam located to the north on Rivers Brook, via a 15-inch diameter concrete pipeline about 920 feet long. Rivers Brook Diversion Dam consists of an earth embankment about 170 feet long having a maximum hydraulic height of about 9 feet. Just short of the midpoint of the embankment, there is an uncontrolled concrete ogee spillway 20 feet wide with a crest elevation 3 feet lower than the dam crest. Plans of the dam and reservoir are included as Appendices B2-7, B2-8, and B2-13. Also, refer to Photos C-11A, C-11B, and C-12A.

Normal reservoir surface at the spillway crest is estimated as being only about 130 feet wide by 170 feet long. At the dam crest, area is estimated as only 0.9 acre (170 feet wide by 220 feet long), with maximum storage estimated at no more than 8 acre-feet based on a maximum depth of 9 feet. Since the maximum dam height is less than 25 feet, and the maximum storage is less than 50 acre-feet, Rivers Brook Diversion Dam is not included in the National Dam Inspection Program. The dam was not inspected, but the data already cited is included in this report for information only.

The popular name of the dam is the same as its official name, Norton Brook Dam. The name of the impoundment is Norton Brook Reservoir. The reservoir is aligned along a northeast - southwest axis with the dam located at the southwesterly end.

The dam is built across Norton Brook, a tributary of the Little Otter Creek. About 6,500 feet downstream from the dam, the Little Otter Creek runs under the New Haven-Monkton Road and then runs between two dwellings. The nearest downstream community is Ferrisburg, population estimated at 150, located about 10 miles downstream from the dam on the north side of the Little Otter Creek. Ferrisburg is not an incorporated village, but is simply a post office location together with houses and other structures.

b. Description of Dam and Appurtenances

1) Main Dam

Referring to the overview photo and the various plans and photos in Appendices B and C, Norton Brook Dam is a rolled and compacted earth embankment with a single spillway of the drop inlet type. The tree-covered embankment consists of a dam section, angled slightly upstream at about its midpoint across a natural stream channel, and an adjacent straight dike section. The dam and dike are barely separated by a narrow rock outcrop abutment. The dam section is about 354 feet long by about 34 feet high. The upstream slope is about 2.5H:1V and the downstream slope is about 2H:1V. Top width varies from 4 to 8 feet.

The dike section, immediately to the left of the dam, is about 243 feet long by about 17 feet high. The upstream slope is about 2H:1V and the downstream slope is about 1.5H:1V. Top width is about 4 feet.

Both the dam and dike have a reinforced concrete core wall that penetrates as much as about 12 feet below the original ground surface, but does not reach bedrock. About half of the length of the core wall in just the dam section on the end toward the dike is supported on two rows of wooden foundation piles with a row of steel sheet piling in between, all driven to bedrock no more than about 30 feet below. The steel sheet piling also acts as a cutoff. The thickness and type of foundation soils under the remainder of the dam and dike are unknown.

The drop inlet spillway consists of a straight uncontrolled weir crest on three sides of a covered rectangular concrete spillway structure (22 feet total effective crest length) located about 40 feet upstream of and connected via a service bridge with the crest of the dam section at about its midpoint. A vertical concrete transition drops about 23 feet into a closed

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

NAME OF DAM: NORTON BROOK DAM, ID NO. VT 00102

SECTION 1

PROJECT INFORMATION

1.1 General

a. Authority

The National Dam Inspection Act, Public Law 92-367, August 8, 1972, authorized the Secretary of the Army through the Corps of Engineers to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Gordon E. Ainsworth and Associates, Inc., has been retained by the New England Division to inspect and report on selected dams in the State of Vermont. Authorization and notice to proceed was issued to Gordon E. Ainsworth and Associates, Inc., under a letter from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-80-C-0012 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection

1) Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public, and thus permit correction in a timely manner by non-Federal interests.

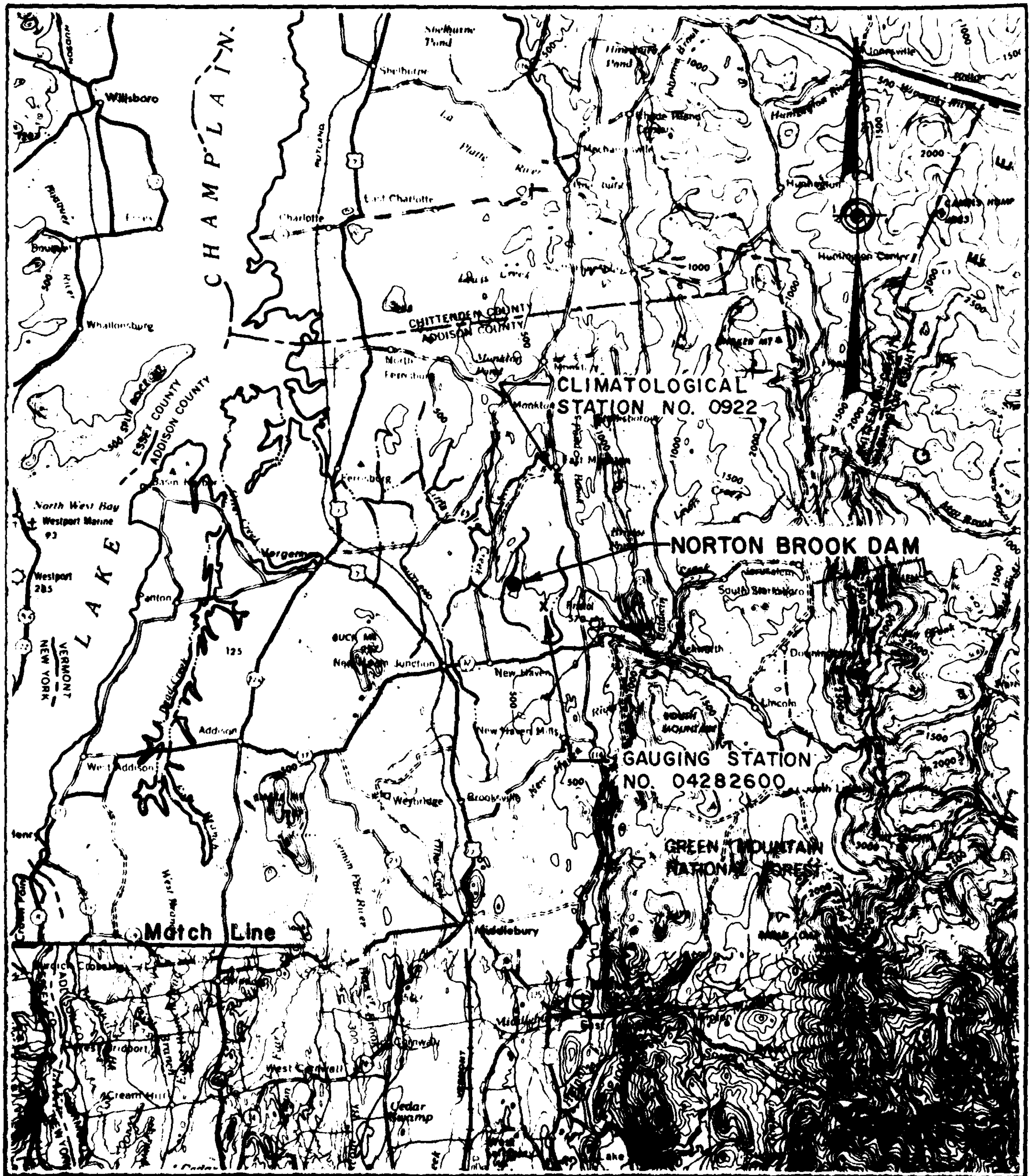
2) Encourage and assist the States to initiate quickly effective dam safety programs for non-Federal dams.

3) To update, verify, and complete the National Inventory of Dams.

1.2 Description of Project

a. Location

Referring to the Location and Vicinity Maps at the beginning of this report, Norton Brook Dam is located in West Central Vermont in the Town of Bristol, Addison County, about 6 miles east of the City of Vergennes. The dam at its maximum section is at Latitude 44 degrees - 9.4 minutes North, Longitude 73 degrees - 8.4 minutes West.



APPROX. SCALE IN MILES

0 2 4 8 12

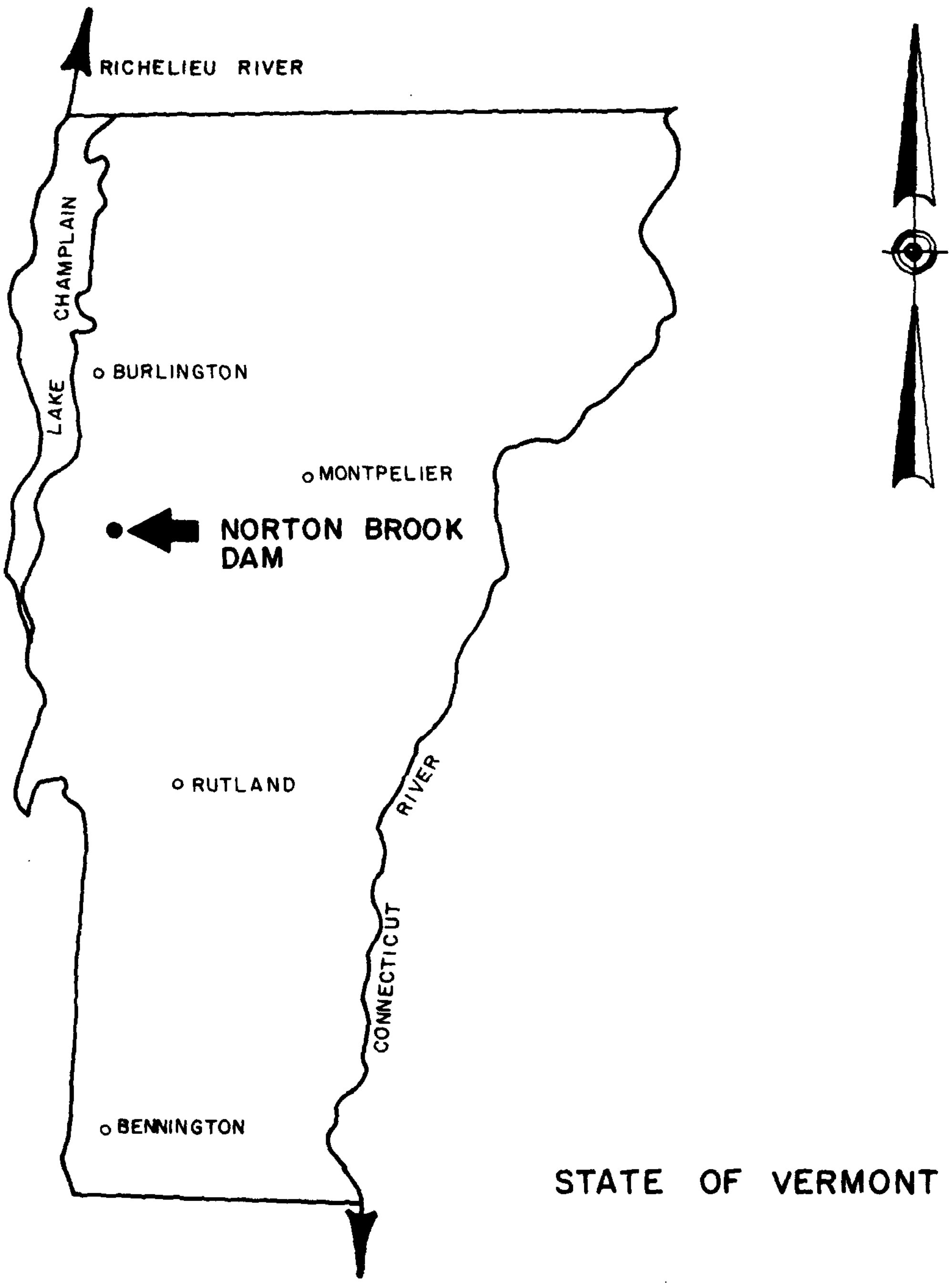
DATUM - NGVD 1929, 100' CONTOUR INTERVAL
BASE MAP - 1:250,000 USGS TOPO MAPS
TOP SIDE - LAKE CHAMPLAIN, UNITED STATES 1947.
BOTTOM SIDE - GLENS FALLS, N.Y., VT., N.H., 1956, LIMITED REVISION 1967

NORTON BROOK DAM VICINITY MAP

GORDON E. AINSWORTH & ASSOCIATES. INC.

Engineers, Surveyors and Planners

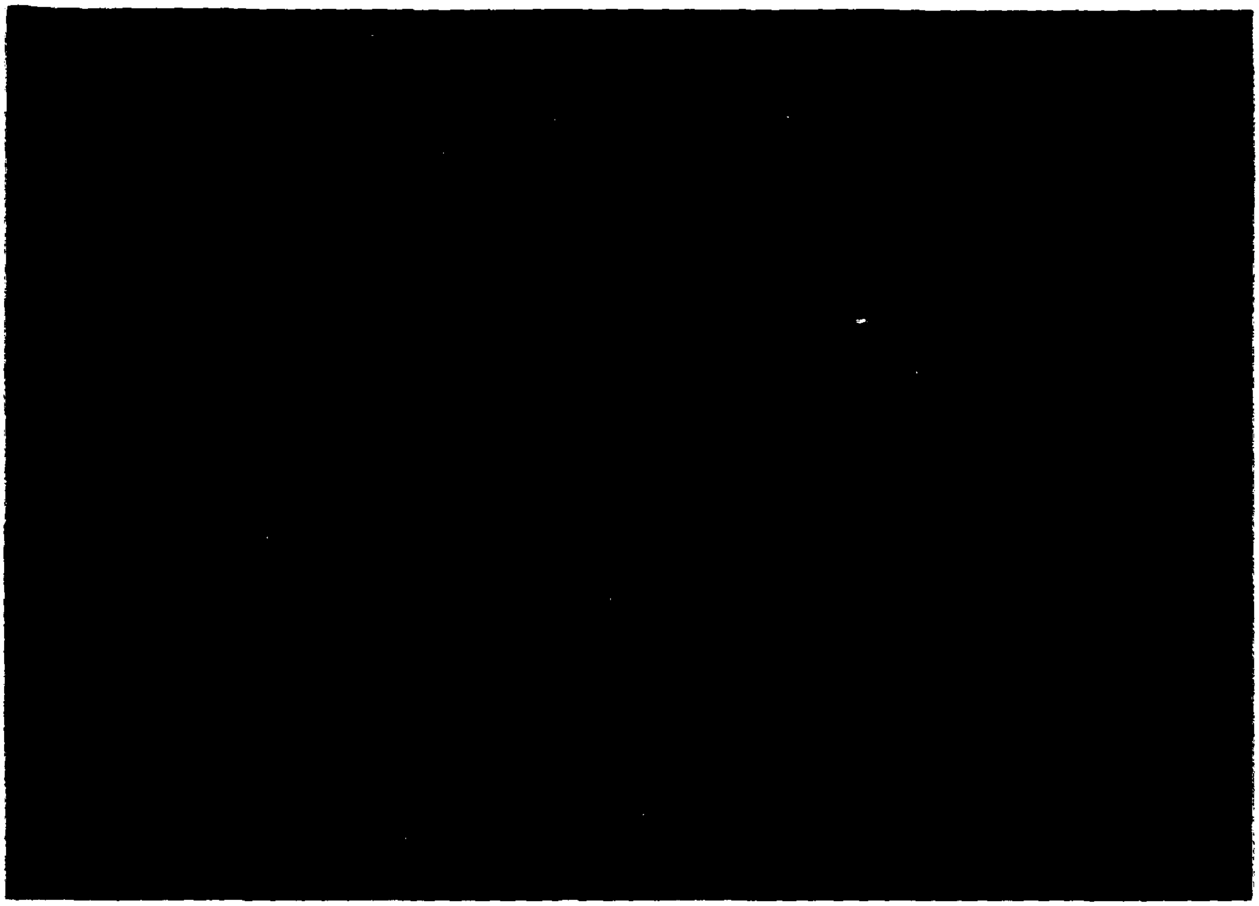
20 SUGARLOAF ST. SOUTH DEERFIELD. MASS. 01373



APPROX. SCALE IN MILES
0 25 50 75

NORTON BROOK DAM
LOCATION MAP

GORDON E. AINSWORTH & ASSOCIATES INC.
Engineers, Surveyors and Planners
20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373



Overview photo - Norton Brook Dam - 11/30/79

c. Impact Evaluation 5-10

6 - EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations 6-1

6.2 Design and Construction Data 6-1

6.3 Post-Construction Changes 6-1

6.4 Seismic Stability 6-2

7 - ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition 7-1
b. Adequacy of Information 7-1
c. Urgency 7-2

7.2 Recommendations 7-2

7.3 Remedial Measures
a. Operation and Maintenance Procedures 7-2

7.4 Alternatives 7-3

APPENDICES

APPENDIX A - INSPECTION CHECKLIST

APPENDIX B - ENGINEERING DATA

APPENDIX C - PHOTOGRAPHS

APPENDIX D - HYDRAULIC AND HYDROLOGIC COMPUTATIONS

APPENDIX E - INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

APPENDIX F - REFERENCES

TABLES

Table 5.1 Overtopping Analysis 5-7

Table 5.2 Dam Failure Analysis 5-9

2) Dike	
a) Crest Movements	3-3
b) Seeps	3-3
c) Trees	3-4
d) Miscellaneous Items	3-4
c. Appurtenant Structures	
1) Intake Structure and Control Tower	3-4
2) Service Bridge	3-5
3) Spillway Structure	3-5
4) Spillway Transition and Conduit	3-6
5) Outlet Structure	3-6
d. Reservoir Area	3-6
e. Downstream Channel	3-6
3.2 Evaluation	3-7
4 - OPERATION AND MAINTENANCE PROCEDURES	
4.1 Operation Procedures	
a. General	4-1
b. Warning System	4-1
4.2 Maintenance Procedures	
a. General	4-1
b. Operating Facilities	4-2
4.3 Evaluation	4-2
5 - EVALUATION OF HYDRAULICS AND HYDROLOGY	
5.1 General	5-1
5.2 Design Data	5-1
5.3 Experience Data	5-1
5.4 Test Flood Analysis	
a. Reservoir Storage	5-2
b. Discharge Capacity	5-3
c. Initial Conditions	5-4
d. Selection of Test Flood	5-4
e. Development of Test Flood	5-5
f. Overtopping Potential	5-6
g. Downstream Analysis	5-6
5.5 Dam Failure Analysis	
a. Failure Conditions	5-6
b. Results of Analysis	5-8

1.3 Pertinent Data

a. Drainage Area 1-6
b. Discharge at Dam Site 1-6
c. Elevation 1-7
d. Reservoir 1-8
e. Storage 1-8
f. Reservoir Surface 1-8
g. Dam - Dike 1-8
h. Diversion and Regulating Tunnel 1-9
i. Spillway 1-9
j. Regulating Outlets
1) Low Level Drain 1-10
2) Water Supply Intake 1-10

2 - ENGINEERING DATA

2.1 Design Data 2-1

2.2 Construction Data

a. Initial Construction 2-1
b. Modifications 2-2
c. Repairs and Maintenance 2-2
d. Pending Remedial Work 2-3

2.3 Operation Data

a. Inspections 2-3
b. Performance Observations 2-4
c. Water Levels and Discharges 2-4
d. Past Floods 2-4
e. Previous Failures 2-4

2.4 Evaluation

a. Availability 2-4
b. Adequacy 2-4
c. Validity 2-5

3 - VISUAL INSPECTION

3.1 Findings

a. General 3-1
b. Dam
1) Main Dam
a) Crest Movements 3-1
b) Seeps 3-2
c) Trees 3-2
d) Miscellaneous Items 3-2

9) Impervious Core - Dam and dike have vertical reinforced concrete cutoff wall, 1 foot thick at top and 3 feet thick at bottom, penetrates up to 12 feet below original ground under dam and up to 5 feet below under dike, but not to bedrock, supported by two rows of wood piles in 120-foot long section of dam only. No pile support under dike.

10) Cutoff - Steel sheet piling in between wood piles in 120-foot long section of dam only. No cutoff under rest of dam or dike.

11) Grout Curtain - None for dam or dike.

12) Other - Concrete core wall under dike was extended to greater depth during repair after washout of dike in June 1942.

h. Diversion and Regulating Tunnel N/A

i. Spillway

1) Type - Upgated Drop Inlet.

2) Length of Weir - Two 8-foot weirs & one 6-foot weir, 22 feet total effective length.

3) Crest Elevation - w/o flashboards 381
- with flashboards N/A

4) Gates - None.

5) Upstream Channel - Not applicable, reservoir all around.

6) Downstream Channel - Spillway discharges into vertical concrete transition 23 feet deep, then through dam via reinforced concrete spillway outlet conduit 101 feet long with horseshoe-shaped cross-section 4 feet wide by 6 feet high, upstream invert EL 358, downstream invert EL 355, then into outlet channel to Norton Brook.

j. Regulating Outlets

1) Low Level Drain

a) Invert - Intake EL 358, Discharge EL 358.

b) Size - 14-inch diameter.

c) Description - Cast iron pipe about 40 feet long discharging to spillway outlet conduit.

d) Control Mechanism - 14-inch spur gear gate valve with handwheel in valve chamber under control tower.

2) Water Supply Intake

a) Inverts - EL 361, EL 369.5 & EL 378.

b) Size - 8-inch diameter.

c) Description - Three 8-inch cast iron intake pipes at different elevations combine in an 8-inch cast iron vertical riser to bottom of valve chamber, then through dam via 8-inch cast iron pipe supported inside spillway outlet conduit, then underground changing to 10-inch cast iron transmission main for about 6 miles to City of Vergennes.

d) Control Mechanism - 8-inch gate valve on each intake pipe with stem to floor stand with handwheel in control tower.

SECTION 2

ENGINEERING DATA

2.1 Design Data

The dam was designed in about 1934 by Barker and Wheeler Engineers, 36 State street, Albany, New York. This firm is no longer in business, and the location of its files is unknown.

The dam and reservoir were part of the design for the entire water supply system for the Owner. The Owner has a complete set of prints of the design/construction drawings. Sheets pertinent to the dam are reproduced at reduced scale in Appendix B2. Included are the original design/construction drawings for the dam, revised design/construction details, and record drawings of construction. The construction specifications are not available.

A review of the design/construction drawings indicates the following geotechnical features that are of some concern:

a. The concrete cutoff wall did not extend to bedrock but extended only a few feet beneath the original ground surface. Thus, water can flow under the wall in all locations except between Sta 0+21R and the left abutment, where sheet piles to bedrock were driven.

b. Under a portion of the cutoff wall for the main dam, Sta 0+21R to the left abutment, steel sheet piling with wooden foundation piles on either side were driven to rock to support the concrete wall. Thus, there is a discontinuity in support of the cutoff wall at Sta 0+21R.

c. No information was given on the character of soils in the foundation beneath the dike.

d. Apparently one row of wood piles was used to support the outlet conduit. (See Appendix B2-9.) Thus, the conduit forms a relatively rigid zone within the deforming embankment, which can lead to transverse cracking due to differential settlements.

2.2 Construction Data

a. Initial Construction

The dam was constructed in 1935 under a PWA grant. The construction work was done by W.G. Fritz Co., 69 Main Street, West Orange, New Jersey. Telephone information has (201) 731-0572 listed for a company of the same name. However, we were

unable to make contact at this number. Therefore, the business status of the original contractor and the location of his files is unknown.

According to the Bid Summary Sheet in the Owner's files, the Fritz Company's bid (second low bid) received on September 28, 1934 was $50,641 for Norton Brook Dam and $8,463 for Rivers Brook Diversion Dam. Both dams appeared as single line items in the total water supply system bid of $145,497.

Appendix B2-13 is a single sheet of record drawings for construction of the dam. It is assumed that all revisions to the design were noted and incorporated into these record drawings.

The Barker and Wheeler Inspection Report of September 20, 1957 (starting on Appendix B3-3) contains some comments on construction and history of the dam, as well as some photographs of dam construction (Appendix B3-10). It is indicated in the report that "... the earth embankment was carefully placed in layers and compacted and rolled...", but that there were foundation problems encountered during construction. Also described at some length was the concern about seepage from the downstream toe during the year after dam construction. The July, 1936 drawing showing seepage locations, which is referenced in the inspection report in the last paragraph on Appendix B3-4, is included as Appendix B2-14. The other plans referenced in the inspection report are also included in Appendix B2.

No other records on the actual construction of the dam are known.

b. Modifications

On Appendix B3-7, there are references to a reported raising of the spillway by as much as 16 inches. This reported raising is cited as a possible contributing cause of the washout of part of the dike on June 21, 1942. There are no known records or details of any spillway raising. It is concluded that any possible spillway raising was by means of flashboards and was only temporary. The design/construction plans (Appendix B2-5) show flashboard slots and sockets on the spillway.

No records of any other modifications to the dam are known.

c. Repairs and Maintenance

After part of the dike washed out on June 21, 1942, the damage was repaired. As noted on Appendix B3-7, the repair work

was done by "... Mr. Overacked of Burlington..." The whereabouts of this gentleman and of any records he may have of his repair work are unknown. Appendix B3-11 shows photos of the washout with repair work underway. It would appear from one of the photos that the core wall was extended deeper as part of the repair. No other records of the repair work are known.

From the inspection reports in Appendix B3, it is documented that brush and undergrowth were cleared off of the downstream slope of the dam in late September and early October, 1957. Appendices B3-12 and B3-13 are photos of the downstream slope taken just prior to the clearing. The clearing was done to allow adequate investigation of settlement of the dam and of seepage at the downstream toe, rather than as a normal maintenance procedure.

On a revised design/construction drawing, Appendix B2-12, there is noted a field observation of May 6, 1959. It concerns joint opening, leakage, and evidence of past repair at the joint between the spillway structure and spillway conduit. No further details of the repair are known.

No further records of past repair and maintenance work are known to exist. Since the Owner has stopped using water from the reservoir in 1972, they have done no maintenance work on dam or reservoir.

d. <u>Pending Remedial Work</u>

The Owner has no plans for any pending remedial work.

2.3 <u>Operation Data</u>

a. <u>Inspections</u>

Only three inspection reports were found, and all are included in Appendix B3. The Inspection Report of September 20, 1957, by Barker and Wheeler Engineers (starting on Appendix B3-3) is notable because it contains comments on construction and history of the dam and on problems associated with the dam. The inspection report is accompanied by four pages of photos starting on Appendix B3-10.

The last documented inspection of the dam appears to have been on October 10, 1957. The short report is presented as Appendix B3-14. The inspection was performed by John Cerutti, Hydraulic Engineer, who apparently conducted the inspection on behalf of the State of Vermont.

b. Performance Observations

Other than the observations on seepage made in the Barker and Wheeler Inspection Report (see Appendix B3-3), there are no other known records of performance observations. There is no instrumentation in the dam.

c. Water Levels and Discharges

There are no known records of routine water levels and discharges from the dam.

d. Past Floods

There are no known records of past floods at the dam.

e. Previous Failures

As noted in the Barker and Wheeler Inspection Report (See Appendix B3-7), on June 21, 1942 part of the dike washed out, exposing and undercutting the core wall. The report indicates that the exact cause of the failure has never been determined. Photos of the washout with repair underway are included as Appendix B3-11. No other records of this failure are known. This is also the only known failure of the dam.

Mr. Carroll Blair, Commissioner of Public Works when the failure occurred, has retired from the City's employ, but still lives in the area. It was indicated by the Owner that Mr. Blair would probably recall more details of the failure and subsequent repair. We were unsucessful in contacting Mr. Blair during our field inspection.

2.4 Evaluation

a. Availability

As listed on Appendix B1, various engineering data and records are available in the files of the Owner and of the Dam Safety Engineer of the Vermont Department of Water Resources. This data was reviewed, and copies of the records significant to the dam are included in Appendices B2 and B3. Discussion of the data starts at the beginning of Section 2 of this report.

b. Adequacy

Available data consisted of the design/construction drawings, the record drawings of construction, and several inspection reports. The design calculations, construction specifications, data on the foundation and embankment soils, and operation and performance data were not available. The lack of such in-depth

engineering data does not permit a comprehensive review. Therefore, the adequacy of this dam could not be assessed with respect to reviewing design, construction, and operation data.

Time permitting, it would be desirable to know more about the construction procedures used in the dam. More data may be able to be found by an in-depth search of the Owner's files. Also, it is believed that the files of the original designer, Barker and Wheeler Engineers, may have been taken over by, and may still exist with, the engineering firm of J. Kenneth Fraser & Associates, P.C., 600 Washington Ave., Rensselaer, New York 12144. The Construction contractor, W. G. Fritz Co., may still be in business in West Orange, New Jersey, and might have some records available. Finally, an interview with the former Commissioner of Public Works for the City, Mr. Carroll Blair, could provide some details on the failure of the dike in 1942, as well as some construction and experience data on the dam.

c. Validity

Based on field observation and checking, the data available appears to be valid. The only discrepancy noted is in the length of the spillway outlet conduit. The design/construction drawings show a length of 101 feet, and this is consistent with the scaled length on the record drawings. One of the revised design/construction drawings (Appendix B2-12) shows a length of 88 feet. The length of the conduit was *not* field checked due to the amount of water flowing in the conduit. However, 101 feet has been used as the length throughout this report.

SECTION 3

VISUAL INSPECTION

3.1 Findings

a. General

Norton Brook Dam was inspected on October 24, 1979. The inspection party (see Appendix A-1) was accompanied by two representatives of the Owner, Mr. Kenneth Thiess, City Manager, and Mr. Carroll O'Connor, Supervisor of Public Works. The weather was overcast, with drizzle, temperature about 55° F. The water surface was at about EL 381.2, about 0.2 of a foot over the crest of the drop inlet spillway. The visual inspection checklist is included as Appendix A, while selected photos taken during the inspection are included as Appendix C. Appendix C-1 is a photo index map. The Overview Photo at the beginning of this report as well as several of the photos in Appendix C are aerial photos taken from a helicopter on November 30, 1979.

This dam is in poor condition. Substantial downward movements of the shells relative to the core wall are evident along the crest. Trees cover the entire downstream slope, crest, and upstream slope above the water level. A mushy zone exists near the right abutment downstream from the toe. A beaver pond downstream obscures seepage that may be occurring in some zones downstream.

b. Dam

1) Main Dam (Between Right Abutment and Rock Outcrop)

a) Crest Movements

Downward movements of the crest of the upstream and downstream shells relative to the central core wall have occurred between Sta 0+50L and 1+30L. (Sta 0+00 is at the angle point in the dam). The settlements are about one foot along this entire zone. The movements apparently were present in 1957, as may be seen on Appendix B3-12 (Barker and Wheeler Inspection Report) in the top right photograph of the dam looking southeast. In a plan view on Appendix B2-13 (record drawings of construction dated January 1936) it is seen that the settlements have occurred along the left half of the portion to the left of the angle point. The depth to bedrock in this zone is about the same or less than it is to the right of the zone that has settled. Therefore, it appears that the foundation soils were most compressible in the zone near the left abutment (i.e., near the rock outcrop).

A depression on the upstream side of the core wall was observed at Sta 1+50R. It is about 1 to 1.5 feet deep and roughly 4 feet in diameter.

b) Seeps

On the natural ground just downstream from the toeline from Sta 1+50R to the outlet structure (Sta 0+25R) there is a wet and somewhat soft zone.

Downstream from the dam there is a beaver pond which covers the downstream portion of the toe to the left of the outlet structure. (See Photos C-2B and C-9A.) This pond obscures any seeps that may exist in this zone and should therefore be removed to allow proper inspection.

Near the left abutment, where the beaver pond does not cover the toe area, two seeps were observed. One is at Sta 1+15L at the left abutment contact and is exiting about 3 feet in elevation above the toe. It was running clear at less than 1/2 gpm. Photo C-9B shows this seep and the eroded area from which it emerges. A second one is 8 feet to the right and is running clear at about 2 gpm. Rusty colored staining has developed downstream from both seeps. A soft zone about 15 feet by 15 feet in size exists at and just above the downstream toe at about Sta 1+00L. The seeps noted above run into this soft zone.

c) Trees

The entire crest and downstream slope is planted with white pine trees. (See photo C-2B.) Photos C-3A and C-3B show the smaller trees that are nearer the crest and on the exposed portion of the downstream slope. Photo C-4A shows the trees and brush that are growing near the left abutment. Photo C-4B shows the larger trees (10 to 12-inch diameter) that grow on the lower part of the downstream slope.

d) Miscellaneous Items

One animal hole, 6-inch diameter, was found in the downstream face. Beavers have eroded paths over the dam, which are sources of potential erosion due to overland flow. Although the paths are well-developed, erosion has not developed significantly.

The riprap on the upstream face is overgrown with brush and trees so that it is difficult to observe. However, a detailed look indicates that most of the face is riprapped beneath the vegetation. Below the water level the riprap appears to be in good condition.

2) Dike (From Rock Outcrop to Left Abutment)

a) Crest Movements

At Sta 2+80L there is a sinkhole-like subsidence on the upstream side of the core wall. The subsidence is about 3 feet deep at the center, as shown in Photo C-10A. It occupies a 15-foot long zone along the core and reaches close to the reservoir shoreline about 6 feet upstream.

On the downstream side of the core wall, between Sta 1+60L and 2+65L the crest has settled 1 to 2 feet relative to the core wall. Photo C-10B shows this relative movement. Mr. Kenworthy is standing on the core wall and the bottom of the rule is on the crest. The side of the core wall is exposed at many locations along this zone, which extends from the right abutment (rock outcrop) to the middle of the dike.

The above-noted movements are in the zone where a washout occurred in this dike in June 1942. The washout is shown in two photos on Appendix B3-11 (Barker & Wheeler Report, Sept. 20, 1957). This washout apparently was caused by water flow under the core wall, which had not been carried to bedrock. The core wall seems to have been extended deeper after the washout, as shown in one of the photos, but it is not known what materials remain beneath. It is significant that the core wall remained intact after the washout, which means either (1) the water level in the reservoir was too low to break the wall by the time the downstream shell had washed out, or (2) the core wall was able to withstand the pressure of the reservoir without the downstream shell in place. There are no details available on the nature of the repair in the washed-out zone.

In a series of photographs taken in September 1957 looking southeast (Barker & Wheeler, Sept. 20, 1957, and included on Appendix B3-12), the second photo down on the left shows the crest of the dike. It is evident in this photograph that the crest of the downstream shell was lower than the top of the core wall by about 1.5 feet in 1957, 15 years after the repair of the washout. The settlement that has occurred since 1957 (22 years) apparently has been small compared with that which occurred during the first 15 years.

b) Seeps

There were no seeps observed on the downstream side of the dike. However, the area is so overgrown with brush, grass, and trees that any seeps are virtually undetectable. Mr. Robert Wheeler, in his letter dated September 20, 1957, (see Appendix B3-6) indicated that seepage at a rate of as much as

about 14 gpm (20,000 gpd) was observed at the right abutment contact in July 1936, prior to the washout. The dike was repaired after the 1942 washout, and on Sept. 10, 1957, Mr. Wheeler reported that the downstream shell was more saturated than he ever recalled. A corrugated steel pipe apparently had been installed at the toe of the dike to take the seepage. Neither the pipe nor the seepage were apparent on the date of this inspection (October 24, 1979).

c) Trees

The slopes and crest of the dike were covered with trees, grass, and shrubs similar to the main dam.

d) Miscellaneous Items

The riprap on the upstream face could not be found between Sta 2+10L and 2+80L. (The sinkhole-like feature is at Sta 2+73L to 2+87L.) The remainder of the riprap appeared to be present.

c. Appurtenant Structures

1) Intake Structure and Control Tower

The intake structure is just upstream of and integral with the control tower. (See Photos C-2A and C-5A.) Most of the intake structure is below the water surface and is not observable. What could be inspected appeared in good condition. (Refer to inspection checklist on Appendix A-5.)

The control tower is pictured in Photos C-2A, C-5A, and C-5B. The inspection checklist is on Appendix A-6. Overall, the control tower appeared in good structural condition. Vandals have broken the windows and doors. In the cast-in-place concrete valve chamber underneath the control tower, there is some efflorescence on the walls, but the concrete appeared sound and showed no actual seeps. (See Photos C-7A and C-7B.) The water intake piping and valves appeared sound, but rusted. The low level and middle level intake valves appeared partially open. The high level floor stand with handwheel has been broken off by vandals and thrown to the bottom of the valve chamber, where it can be seen lying in the lower right of Photo C-7B. Photo C-7B also shows the low level outlet in the center, a 14-inch spur gear gate valve, and what are reported to be 4-inch valve chamber drain valves, or backwater valves, with handwheels on each side. The operable condition of all the valves is unknown.

2) Service Bridge

The service bridge is a concrete-decked walkway sloping slightly upward from the dam crest over an intermediate pier to the spillway structure. The flat top of the spillway structure completes access to the control tower. (See Photos C-2A, C-5A, and C-5B.) The inspection checklist is on Appendix A-10.

Other than some of the supporting steel for the bridge being rusted, the bridge is in good condition. Referring to Photo C-6A, there is what appears to be ice damage to the concrete at the waterline on the legs of the intermediate pier. From the design/construction drawings, Appendix B2-5, it appears that the pier legs are steel H-columns encased in mesh reinforced concrete. The concrete encasement should be repaired.

Photo C-6B shows a crack about 3/4 inch wide across the end of the service bridge at the abutment on the dam crest. From the design/construction drawings, Appendix B2-4, it appears that the service bridge rests on a seat cast into the top of the core wall. The top of the core wall is shown extending upward past the end of the bridge deck to be level with the deck, i.e., it would be the concrete to the left of the crack in Photo C-6B. The service bridge appears to be bolted tight to the spillway structure. Hence, any expansion, contraction, or other movement of the service bridge has to appear in the crack at the abutment on the dam crest. The crack should be watched and investigated further to confirm its suspected function as an expansion joint. If it is in fact an expansion joint, it should be filled with an expansion joint material to prevent water entry and ice action.

3) Spillway Structure

The drop inlet spillway structure is just downstream of and integral with the control tower. (See Photos C-2A and C-5A.) There is a spillway weir on three sides of the structure, separated by corner posts, with the fourth side of the structure common with the control tower and underlying valve chamber. (See design/construction plans, Appendix B2-5.) The inspection checklist is on Appendix A-9.

The spillway weir was difficult to inspect due to flowing water. There appears to be minor spalling of the concrete at the corner posts. Flashboard sockets and a metal weir plate as indicated on the plans (Appendix B2-5) were not observed due to the flow of water.

4) Spillway Transition and Conduit

The spillway outlet conduit was difficult to inspect due to flowing water, and the vertical transition shaft was impossible to inspect for the same reason. The inspection checklist is on Appendix A-7.

The spillway conduit contains a vertical crack on the side walls about 26 feet upstream from the downstream end, as well as near the center of the dam. These cracks may be due to differential movement between the core wall at the center and the concrete conduit. Also, the conduit may have settled differentially if the pile support was insufficient.

5) Outlet Structure

The inspection checklist is on Appendix A-8, while Photo C-8A shows the outlet structure and part of the training walls. The training walls of the outlet channel have cracked and tipped toward the channel at the top, apparently due to frost action. (See Photo C-8B.) This process can be expected to lead to collapse of segments of the wall.

d. Reservoir Area

There does not appear to be excessive reservoir sedimentation. No potential landslide areas were noted around the reservoir. Also, there does not appear to be any potential hazard due to backwater flooding of the reservoir. No specific detrimental features were observed that might cause excessive alteration of the drainage area or increased inflow. There appears to be a security fence (6-foot high wire topped with 1 foot of barbed wire) all around the dam and reservoir. However, the fence is in poor condition with gaps that would allow easy access.

e. Downstream Channel

Immediately downstream of the outlet structure the discharge channel has been flooded by a beaver pond. (See Photos C-2B and C-9A.)

For a map of the remainder of the channel, refer to Appendix B2-1 as well as the Drainage Area Map, Appendix D-1. Photo C-12B is an aerial view of the reservoir and channel looking downstream, while Photo C-13A is the same area looking upstream at the reservoir.

About 800 feet downstream of the dam, the channel (Norton Brook) crosses the access road to the dam. About 1,300 feet downstream, Norton Brook runs into Rivers Brook. About 1,700 feet downstream, Rivers Brook runs about 350 feet westerly of a fish and game club building (seen in Photo C-13A), and then at about 2,000 feet downstream, crosses under Plank Road. From the dam down to Plank Road, the stream meanders and has brush and trees growing all along its banks. No significant obstruction to flow was observed.

About 3,000 feet downstream, Rivers Brook joins the Little Otter Creek in a wide flat flood plain. This plain downstream of Plank Road can be seen toward the upper left corner of Photo C-12B. The Little Otter Creek then meanders through a narrow shallow valley until it crosses the New Haven-Monkton Road about 6,500 feet downstream of the dam. This channel can be traced along the top of Photo C-12B with Photo C-13B looking upstream at the intersection of Plank Road (top to bottom) and the New Haven-Monkton Road (right to left). In the photo, note the house trailer on the left and the house on the right of the Little Otter Creek between the two roads.

3.2 Evaluation

a. The presence of the sinkhole-like feature on the upstream side of the core wall of the dike and the depression on the upstream side of the core wall of the dam should be investigated. The design of this dam and the fact that a washout occurred previously under the dike core wall both indicate that a similar washout is still possible, particularly under the main dam.

b. The large number of trees and the brush that have been allowed to grow on this dam make it very difficult to observe seeps or signs of movements on the downstream side. The tree roots, although relatively shallow in this case, can create flow paths. For these reasons the slopes should be cleaned of all vegetation except grass and kept that way to a distance of about 20 feet downstream from the toe.

c. A monitoring program should be maintained for all seeps.

d. The operating condition of all the control valves should be determined, particularly the low level outlet valve.

e. Concrete damage on the intermediate pier legs of the service bridge should be repaired. Also, the crack at the end of the service bridge at the abutment on the dam crest should be investigated and modified as a true

rise from 1.1 to 6.1 feet deep, an increase of 5.0 feet, which floods an area about 140 feet wide. Velocity of flow accelerates about 3 times to 13 fps.

At Sta 20+00 near Plank Road and the fish and game club building, peak flow increases about 45 times to 7,600 cfs after about 20 minutes. This causes the water to rise from 1.4 feet to 4.0 feet deep, an increase of 2.6 feet, which floods an area about 650 feet wide. The fish and game club building appears to be outside the limits of flooding.

At Sta 70+00 near two inhabited structures, peak flow increases about 43 times to 7,300 cfs after about 30 minutes. This causes the water to rise from 1.2 feet to 3.4 feet deep, an increase of 2.2 feet, which floods an area about 990 feet wide. Velocity of flow accelerates about 2 times to 4 fps. The two inhabited structures appear to be flooded to a depth of 1 to 2 feet over their first floors due to the increase in flow from the dam breach under test flood conditions.

The flood routing was not carried any further downstream than Sta 70+00 because flood depths were already getting relatively shallow. Also, downstream from Sta 70+00, there are wide flood plains and a scarcity of dwellings near the Little Otter Creek. The nearest downstream community of Ferrisburg is some 8.6 miles further downstream. Between Sta 70+00 and Ferrisburg, it is estimated from a USGS map that there are only 4 structures within 1,000 feet of the Little Otter Creek, and all off these are more than 10 feet above the channel. Also, most of Ferrisburg itself is 3,000 feet from the stream and no structure appears to be less than 30 feet above the stream channel.

Thus, it appears that a major failure of the dam under test flood conditions would impact at least two dwellings, probably damage portions of Plank Road, and flood some farmland next to the Little Otter Creek. There appears to be potential for future development in the impact area. Also, a dam failure would disrupt the water supply for the 17 families that still use water from the reservoir.

Since the peak outflow from the dam failure occurs within the breach development time (i.e., at 16.74 hours within the 16.42 to 16.92 breach development time), the peak outflow and resulting impact area are due directly to the dam failure. They are not due to the flood peak being routed through the breach after the breach is fully developed.

A second dam breach was also modeled with the HEC-1 DB program and is listed last in Table 5.2 as DAM BREACH - NO

the water surface reaches a maximum below the top of the dam and peak flow approaches total project discharge due to the test flood. It is the same as and is taken from the overtopping analysis previously summarized in Table 5.1. Results are summarized only at the more important downstream stations, while the computer input and output for all stations starts on Appendix D-11.

DAM BREACH - TEST FLOOD is a major sudden failure of the dam under test flood conditions using the parameters previously discussed in Section 5.5a. Results are summarized in Table 5.2 for all stations, with the computer input and output starting on Appendix D-27.

From the computer listing and plot of the breach hydrograph on Appendix D-32 and 33, note that the standard calculation interval selected (5 minutes = 0.083 hours) was short enough to permit the interpolated breach hydrograph at the standard time interval to closely approximate the computed breach hydrograph. Only the interpolated breach hydrograph is routed downstream. Also, note that the breach time was long enough to include the peak of the breach hydrograph.

c. Impact Evaluation

For a sudden major dam failure, DAM BREACH - TEST FLOOD, the computed *maximum* water surface elevation for each downstream station is tabulated in Table 5.2 and is plotted on each cross-section beginning on Appendix D-22. The top widths of flow determined from each cross-section are tabulated in Table 5.2 and are plotted on Appendix D-1 to define the limit of the impact area, i.e., the limit of flooding or hazard due to the dam failure. Also, the computed water surface is shown on the channel profile, Appendix D-26.

The average velocity of peak flow (flow divided by total flow area) is also listed in Table 5.2 for each downstream station for all failure cases. The flow area calculation is shown on each cross-section plot starting on Appendix D-22 for only the DAM BREACH - TEST FLOOD case.

Just prior to the dam breach, flow from the dam and at downstream stations was approaching 170 cfs, the total project discharge due to the test flood not overtopping the dam. Flow at the first station 800 feet downstream was about 1.1 feet deep at about 4 fps. Approximately 19 minutes (0.32 of an hour) after the breach starts, peak outflow from the dam increases about 46 times to 7,800 cfs. This causes water 800 feet downstream to

TABLE 5.2

NORTON BROOK DAM

DAM FAILURE ANALYSIS

CONDITIONS — Same as Overtopping Analysis, Table 5.1
Start Routing at Spillway Crest Elev. 381, Dam Crest Elev. 385
Total Project Discharge Capacity at Dam Crest 540 cfs ±
Due to Spillway only. Outlet Works Closed.

	Approx. Peak Flow (cfs)	Time to Peak Flow (hours)	Approx. Max. Water Surface Elev. (feet)	Depth (feet)	Top Width (feet)	Avg. Vel. (fps)
NO BREACH - TEST FLOOD						
Total Project Discharge due to Test Flood - Dam not Overtopped						
Dam	170	16.58	382.78	27.8	---	--
Sta 8+00	170	16.58	349.1	1.1	60	4
Sta 20+00 Near Plank Road	170	16.75	339.4	1.4	230	3
Sta 30+00	170	16.83	334.5	1.5	175	3
Sta 37+00	170	16.92	334.0	3.0	20	4
Sta 70+00 Near Dwelling	170	17.08	279.2	1.2	240	2
DAM BREACH - TEST FLOOD						
Start Breach W.S. at 382.78						
Time of Failure = 16.42 hours						
Breach Time = 0.50 hour						
Breach Width = 90 feet						
Breach Depth = 30 feet						
Trapezoid, 0.5H : 1V side slopes						
Dam	7,800	16.74	382.78	27.8	---	--
Sta 8+00	7,800	16.75	354.1	6.1	140	13
Sta 13+00	7,800	16.75	350.0	8.0	250	10
Sta 20+00 Near Plank Road	7,600	16.75	342.0	4.0	650	5
Sta 25+00	7,600	16.83	340.8	4.8	880	4
Sta 30+00	7,700	16.83	337.3	4.3	1025	4
Sta 37+00	7,400	16.83	344.7	13.7	180	7
Sta 55+00	7,300	16.92	304.5	6.5	130	16
Sta 65+00	7,400	16.92	286.7	6.7	85	19
Sta 70+00 Near Dwelling	7,300	16.92	281.4	3.4	990	4
DAM BREACH - NO FLOOD						
Start Routing and Breach W.S. at Spillway Crest						
Time of Failure = 0.0 hour						
Same Breach Conditions as for Test Flood						
Dam	7,100	0.33	381.0	26.0	---	--
Sta 20+00 Near Plank Road	6,700	0.42	341.8	3.8	645	5
Sta 70+00 Near Dwelling	6,100	0.58	281.2	3.2	985	4

call for breaching the dam when the water surface reaches the dam crest due to the test flood, or reaches the maximum water surface elevation due to the test flood when the test flood does not overtop the dam. Since the test flood of one-half PMF does not overtop the dam, the dam breach was allowed to begin when the water surface reached the maximum elevation due to the test flood, EL 382.78, about 2.2 feet below the dam crest. The inflow test flood and the contents of the reservoir were routed through the dam breach as the breach progressed. All other routing conditions and test flood development were the same as for the overtopping analysis previously discussed.

To model a sudden major dam breach, maximum breach geometry was selected as follows: constant trapezoidal shape with 0.5H:1V side slopes, breach width across the bottom of the trapezoid equal to 40% of the length of the dam at mid-height (225 x 0.4 = 90 feet), and a breach depth below the top of the dam equal to 30 feet, which approximates a full depth failure that would completely drain the reservoir. In addition, a minimum breach time, or time for the breach width to progress the full depth from the top to the bottom of the dam, of 0.5 hours was selected to model a sudden failure.

The inputted cross-sections defining the downstream channel reaches were developed from and are located on the USGS map included as Appendix D-1. Hand plottings of the cross-sections start on Appendix D-22, while Appendix D-26 is a profile of the downstream channel. Normal depth channel routing was performed by the HEC-1 DB program using the Manning's n values for left overbank, channel, and right overbank as listed on each cross-section plot. The overbank points and the actual channel section in between are only an approximation of the true natural channel. This is because of the constraints of the small scale USGS map that the cross-sections were developed from and of the limited 8-point cross-section accepted by the program. The third and sixth point on each cross-section are defined as the overbank points. Therefore, distinguishing between in-channel and overbank flow cannot be done reliably by simple comparison of computed water surface depth with the defined overbank points. It must be done by judging the calculated quantity, depth, width, and velocity of flow against the real channel cross-section and configuration as it exists. All the cross-sections are the same as those used for the downstream analysis without any dam breach as referred to in Section 5.4g.

b. Results of Analysis

The results of the dam failure analysis using the HEC-1 DB program are summarized in Table 5.2. NO BREACH - TEST FLOOD approximates downstream conditions just prior to a breach, as

TABLE 5.1

NORTON BROOK DAM

OVERTOPPING ANALYSIS

CONDITIONS — Total Drainage Area = 0.155 Square Mile
Plus Inflow from Rivers Brook Diversion Dam = 7 cfs
Start Routing at Spillway Crest Elev. 381, Dam Crest Elev. 385
Total Project Discharge Capacity at Dam Crest 540 cfs ±
Due to Spillway only. Outlet Works Closed.
Some Values Rounded from Computed Results.

		TEST FLOOD ONE-HALF PMF (a)
INFLOW		
24-hour Rainfall (inches)		10.5 (b)
24-hour Rainfall Excess (inches) (c)		7.9 (d)
Peak Inflow	(cfs)	330
	(csm)	2,130
OUTFLOW		
Peak Outflow	(cfs)	170
	(csm)	1,100
Time to Peak Outflow (hours)		16.58
Maximum Storage (acre-feet)		197
Max. W.S. Elevation (feet-NGVD)		382.8
Minimum Freeboard (feet)		2.2
Maximum Depth over Dam (feet)		n/a
Duration of Overtopping (hours)		n/a

(a) One-half of full PMF total runoff, including base flow. For one-half PMF base flow = 2 cfs per square mile = 1 cfs ± for the land surface, plus 7 cfs diversion inflow.
(b) Approximation assuming total losses are the same as for the full PMF. Full PMF 24-hour rainfall equals 18.5 inches.
(c) Rainfall Excess = Rainfall for the Reservoir Surface. For the rest of the drainage area, losses are assumed to be 1.0 inch initially and 0.1 inch per hour thereafter.
(d) Equal to one-half of full PMF value. Full PMF 24-hour rainfall excess for the land surface equals 15.9 inches.

conservative standard lag time was used. The program uses the inputted Snyder coefficients to solve by iteration for approximate Clark coefficients, which are then used to calculate the runoff hydrograph.

For the reservoir surface making up Sub-area 2, loss rates were set to zero so that rainfall would equal rainfall excess, or runoff. Assuming no delay in the rainfall/runoff response, a constant unit hydrograph for a rainfall duration equal to the HEC-1 DB calculation interval was developed per Appendix D-10 and inputted to the model.

f. Overtopping Potential

The results of the overtopping analysis using the HEC-1 DB program are summarized in Table 5.1. The overtopping analysis computer input and output for the test flood of one-half PMF are included starting on Appendix D-11.

As noted from Table 5.1, the test flood of one-half PMF does not overtop the dam, but results in a minimum freeboard of about 2.2 feet. Peak inflow for the test flood is 330 cfs, or 2,130 csm (cfs per square mile). Peak outflow is reduced substantially by reservoir routing to 170 cfs, or 1,100 csm, and occurs about 17 hours after the start of the storm. The peak portion of the inflow and outflow hydrograph for the test flood (one-half PMF) is shown by the computer plot on Appendix D-16. Total project discharge capacity at the top of the dam is due only to the drop inlet spillway (outlet works assumed closed) and is equal to 540 cfs, or 318% of the test-flood peak outflow.

g. Downstream Analysis

Not summarized in Table 5.1, but included in the computer input and output starting on Appendix D-11 are the results of normal depth channel routing of the flood outflow through the downstream channel reaches. The downstream cross-sections are located on Appendix D-1. The cross-sections and routing methods are the same as those used for the dam failure analysis in Section 5.5 of this report which follows immediately. The calculations are used to approximate conditions downstream just prior to a hypothetical dam failure as discussed in Section 5.5.

5.5 Dam Failure Analysis

a. Failure Conditions

In order to evaluate the downstream hazard, the flood flow due to a major failure or breach of the dam was routed downstream using the HEC-1 DB program. Corps of Engineers' criteria

area, the test flood selected for this evaluation was one-half PMF (peak inflow 330 cfs, peak outflow 170 cfs, per Table 5.1).

The PMF event is that hypothetical flood flow produced by the most critical combination of precipitation, minimum infiltration loss, and concentration of runoff that is considered reasonably possible for a particular drainage area.

e. Development of Test Flood

The U.S. Army Corps of Engineers Hydrologic Engineering Center's Program HEC-1 DB (Reference 3) was used to develop the test flood hydrology and perform the reservoir routing. The index PMP (probable maximum precipitation) inputted to the computer model was 17.5 inches for a 24-hour duration all-season storm over a 200-square mile basin, according to HMR 33 (Reference 4). Maximum 6-hour, 12-hour, and 24-hour percipitation for the actual size of the drainage area (same for 10 square miles or less) were inputted to the model as percentages of the index PMP in accordance with HMR 33. A storm reduction coefficient was then applied internally by the program in order to transpose or center the storm over the actual total drainage area. Thus, the corrected 24-hour PMP for the actual total drainage area became 18.5 inches.

In accordance with accepted practice, floods as ratios of the PMF (e.g., one-half PMF) were taken as ratios of runoff, not of precipitation. The HEC-1 DB program applies the ratio to total runoff, including base flow. This method of applying the ratio introduces an increasing error in base flow as the ratio of the PMF gets smaller. However, this error was eliminated by inputting twice the desired base flow to the full PMF, so that one-half PMF, the test flood, would have the correct base flow.

All precipitation was distributed by the program using the Standard Project Storm arrangement of EM-1110-2-1411 (Reference 13), including the percentage distribution for the maximum 6-hour precipitation, and by both the arrangement and percentage distribution from HYDRO-35 (Reference 6) for the maximum 1-hour precipitation.

Appendix D-10 summarizes the sub-area, loss rate, and unit hydrograph data inputted to the program. Only two sub-areas were used. Sub-area 1 consists of all the drainage area around the reservoir, and Sub-area 2 consists of just the reservoir surface. For the land in Sub-area 1, loss rates were assumed to be 1.0 inch initially and a constant 0.1 inch per hour thereafter. Snyder unit hydrograph parameters were assumed for average conditions per Appendix D-10 and inputted to the program. A

With the reservoir at the dam crest, EL 385, 4 feet over the spillway crest, the total discharge from the dam is 540 cfs. This is due solely to the drop inlet spillway. Also, with an average discharge of 270 cfs over the 4-foot depth from the top of the dam down to the spillway crest, it would take about 2.8 hours for the spillway to drain the 63 acre-feet of storage between the top of the dam and the spillway crest, or about 0.7 of an hour per foot, all assuming no inflow.

c. Initial Conditions

The purpose of this analysis is to evaluate the dam and spillway with respect to the adequacy of their surcharge storage and spillway capacity. Accordingly, it was assumed that the water surface was at the spillway crest at the start of the flood routing. For all conditions, the low level outlet or drain pipe was assumed closed, as it is normally, and the water main intakes were also assumed closed, since the reservoir is no longer used as an active water supply.

Since the exact status of the 15-inch diameter pipeline from Rivers Brook Diversion Dam is not clear, it was assumed to be fully open and discharging a constant 7 cfs to Norton Brook Reservoir for the duration of the flood routing. Appendix D-9 shows the calculation of the 7-cfs inflow from the diversion dam pipeline. The inflow is strictly a function of the hydraulic capacity of the pipeline created by the difference in head between the water surfaces in the diversion dam and in the reservoir. For simplicity, the difference in head was assumed constant throughout the flood routing and equal to the difference when both the reservoir and the diversion dam were level with their respective spillway crests. The 7 cfs was inputted to the program as a constant base flow into the reservoir, Sub-area 2.

A constant base flow of 2 cfs per square mile to represent average conditions was also inputted for the land surface, Sub-area 1.

d. Selection of Test Flood

Based on the dam failure analysis presented later in Section 5.5, Norton Brook Dam is classified as having a significant hazard potential (two dwellings, one road, and some farmland). Since the dam is also classified as small in size (see Section 1.2c), recommended guidelines of the Corps of Engineers (Reference 1) indicate a test flood in the range of the 100-year flood to one-half PMF (probable maximum flood). Since the dam is near the upper end of its small size range with regard to height, and since there is potential for future development in the hazard

b. Discharge Capacity

The only spillway for the dam is a single drop inlet structure. Referring to the design/construction plans in Appendix B2, the spillway consists of a straight weir crest on three sides of a covered rectangular concrete outlet structure (22 feet total effective crest length by 3.5-foot high rectangular entrance), a vertical concrete transition dropping about 23 feet, and a closed concrete spillway outlet conduit about 101 feet long with 3 feet of drop, and having a horseshoe-shaped cross-section 4 feet wide by 6 feet high.

The discharge capacity of each of the three spillway weirs was conservatively calculated assuming that its entrance acted as a rectangular sharp-crested weir with end contractions up to and including full entrance flow at 3.5 feet of depth. For water depths greater than 3.5 feet, orifice flow through the entrance was assumed. Total spillway capacity was taken as the sum of the spillway capacities of the three spillway weirs. The spillway capacity calculations are presented as Appendix D-5. With water 4 feet over the spillway, the spillway discharges a total of 540 cfs.

The approximate full flow capacity of the spillway outlet conduit was calculated as 840 cfs per Appendix D-6. The calculations are based only on Manning's Equation for open channel flow with free discharge. Any reduction of capacity due to end losses or high tailwater has been neglected. Similarly, any increase in capacity due to pressure flow because of a head buildup in the vertical transition has been neglected.

Taking the spillway crest at EL 381 and the dam crest at EL 385, the spillway discharge computations are summarized on Appendix D-7 and graphed on Appendix D-8. Total discharge from the dam is the sum of the discharges from the drop inlet spillway plus flow over the dam for the overtopping condition. Flow over the dam was computed assuming critical flow over a rectangular broad-crested weir with a level crest length equal to the total length of the dam and dike. The crest elevation, length, approximate discharge coefficient, and exponent of head were inputted to the HEC-1 DB computer program (Reference 3). The formula used for the calculation, as well as the results of hand calculation at selected points, is shown on Appendix D-7.

When flow in the drop inlet spillway reaches the 840-cfs full-flow capacity of the spillway conduit previously calculated, the spillway is considered to be controlled at that conduit capacity for any greater water depths. Note that this does not occur until the reservoir reaches EL 388, or 3 feet over the dam.

According to USGS Water Resources Data (Reference 19), the nearest stream gauging station is No. 04282600 located on a tributary of the Little Otter Creek about 2 miles northwest of Bristol, at Latitude 44 degrees - 08.73 minutes North, Longitude 73 degrees - 07.08 minutes West. The station is at a culvert under Plank Road about 1.7 miles east of where it is intersected by the access road to Norton Brook Dam, and about 3.6 river miles upstream from the confluence of the Little Otter Creek and Rivers Brook. The station has a drainage area of 1.48 square miles and a period of record from 1964 to the present. The station is identified on the Vicinity Map at the beginning of this report and is about 1.2 miles southeast of Norton Brook Dam.

According to NOAA Climatological Data for New England (References 20 and 21), the nearest climatological station is No. 0922, Bristol 5 NNW, located near East Monkton about 5 miles north-northwest of Bristol, at Latitude 44 degrees - 12 minutes North, Longitude 73 degrees - 07 minutes West. The station is non-recording, and temperature and precipitation observations are made. Years of record start in about 1965. The station is identified on the Vicinity Map at the beginning of this report and is located about 3.4 miles northeast of Norton Brook Dam.

5.4 Test Flood Analysis

a. Reservoir Storage

Using a bathymetric map of the reservoir from the original design/construction plans (Appendix B2-2), supplemented by existing USGS contour mapping (Appendix D-1) above the dam crest, areas inside contour elevations were measured and the capacity of the reservoir was calculated using the method of conic sections. The calculations were done both by hand (Appendix D-2) and by the HEC-1 DB computer program (Reference 3) with results of computer calculation on Appendix D-14. Hand and computer calculations agree, with the calculated volume for the 20 feet of reservoir below the spillway crest agreeing within 7% of the volume reported on the design/construction plans (164 acre-feet calculated vs. 153.4 acre feet or 50 million gallons reported).

Using the calculated values, elevation-area and elevation-storage curves are presented on Appendices D-3 and D-4 respectively. At the drop inlet spillway crest, EL 381, the reservoir has a surface area of 14.7 acres and a total capacity of 170 acre-feet. At the dam crest, EL 385, the surface increases to 17.0 acres and the capacity to 233 acre-feet, or about 76 million gallons. Surcharge storage between the spillway crest and the dam crest amounts to 63 acre-feet, or about 7.6 inches of runoff from the 0.155 square-mile drainage area. Therefore, the reservoir has a substantial capacity to attenuate peak inflow.

SECTION 5

EVALUATION OF HYDRAULICS AND HYDROLOGY

5.1 General

Norton Brook Dam is shown on the Location and Vicinity Maps at the beginning of this report and on the Drainage Area Map, Appendix D-1. The dam and reservoir are at the headwaters of Norton Brook in west central Vermont. About 1,300 feet downstream of the dam, Norton Brook runs into Rivers Brook, which then joins the Little Otter Creek about 3,000 feet from the dam. The Little Otter Creek then meanders northwesterly about 10 river miles to Lake Champlain. Lake Champlain is drained by the Richelieu River northerly into Canada.

The total drainage area at the dam is about 0.155 square miles, of which about 0.023 square miles (14.7 acres), or 15%, is actual reservoir surface at the spillway crest elevation. Being in the northwestern foothills of the Green Mountain National Forest, the topography is characterized by fairly steep wooded slopes averaging 25%. Elevations in the drainage area vary from EL 381 to EL 847.

The reservoir can also receive additional inflow (estimated at 7 cfs) via a pipeline about 920 feet long from Rivers Brook Diversion Dam to the north. The total drainage area at the diversion dam is 0.976 square miles, or over 6 times more than the area naturally tributary to Norton Brook Dam.

5.2 Design Data

There are no known records of the hydraulic and hydrologic criteria used in the original design of the dam and reservoir. Other engineering data available, mainly the original design/construction plans, are discussed in Section 2 of this report.

5.3 Experience Data

As noted in Section 2.3 of this report, there are no known records of routine water levels and discharges or of past floods at the dam. On June 21, 1942, part of the dike did fail, but the failure appears to have been by washout undercutting the core wall due to deliberate reservoir raising rather than by dam overtopping. Although not reported in the available engineering data, a storm event may have contributed to the failure. What limited records there are of this failure are discussed in Sections 2.2c and 2.3e of this report.

The only maintenance currently being done is watershed management. This is essentially tree thining work done by the County Vocational Agency under an agreement with the City of Vergennes. The City owns most, if not all, the property comprising the watershed of the dam and reservoir, including that tributary to Rivers Brook Diversion Dam.

b. Operating Facilities

(Covered under preceding Section 4.2a - General.)

4.3 Evaluation

Operation and maintenance procedures for this dam do not exist. There has been no effort to operate or maintain the dam since 1972. Effective operation and maintenance procedures need to be developed and implemented by the Owner in order to avoid worsening deterioration of the dam. A reservoir regulation plan should be developed as part of the operation procedures.

A warning system with an emergency action plan needs to be developed by the Owner to insure proper and timely action during critical periods.

SECTION 4

OPERATION AND MAINTENANCE PROCEDURES

4.1 Operation Procedures

a. General

Norton Brook Reservoir has not been used as a water supply for the City of Vergennes since 1972. Since that time, the City has drawn water from Lake Champlain. Since 1972 the City has essentially abandoned the operation and maintenance of the dam and reservoir. Consequently, there are no current operation procedures for the dam and reservoir.

The drop inlet spillway is uncontrolled and wide open, and is left that way. The high level, middle level and low level water main intakes appear to be partially open. This allows the 17 families who live along the route of the transmission main, and who are still tapped into the main, to use raw water from the reservoir. The City would like to shut the water main intake valves entirely, but the 17 families still using the water presently have a suit against the City trying to force the City to continue to provide them with water.

The 15-inch diameter pipe feeding the reservoir from Rivers Brook Diversion Dam appears to be valved off inside the outlet works at the Diversion Dam. However, the exact status of this pipe and valve are not known. The valve could not be inspected because the valve pit at the diversion dam was flooded with water. The ogee spillway at the diversion dam is uncontrolled and wide open and discharges water to Rivers Brook, thus bypassing water completely around Norton Brook Reservoir.

b. Warning System

There is no warning system in effect for Norton Brook Dam.

4.2 Maintenance Procedures

a. General

Since the City of Vergennes stopped using water from Norton Brook Reservoir in 1972, they have done no maintenance work on the dam or reservoir. Consequently, there are no current maintenance procedures for the dam and reservoir and their operating facilities.

expansion joint if that is its function as is now suspected.

f. The crack in the spillway conduit about 26 feet from its downstream end should be repaired. Continue to watch the existing cracks and displacements in the conduit for signs of change.

g. The cracked and tipped training walls of the outlet channel should be repaired.

h. The beaver dam in the downstream channel should be removed to eliminate ponding back to the downstream toe.

FLOOD. In accordance with Corps of Engineers' criteria, this is a static, sunny-day failure, modeled with no inflow flood or base flow, and with both routing and breaching starting at the spillway crest at time zero. All other conditions, including breach geometry, breach time, and downstream routing parameters are the same as for the test flood dam breach. Downstream conditions prior to failure are assumed to be zero flow, zero depth. Therefore, the no-flood dam breach indicates the severity of a dam breach flood unaffected by stream flood or base flow conditions.

Results for DAM BREACH - NO FLOOD are summarized in Table 5.2 for selected stations, with the computer input and output for all stations starting on Appendix D-39. At Sta 70+00, where the two inhabited structures are located downstream of the dam, note that the no-flood breach causes a depth of flow, i.e., an increase in depth over zero prior flow, of 3.2 feet. This is more than the 2.2-foot increase (1.2 to 3.4 feet) due to the test flood breach over prior no-breach, test flood discharge conditions without overtopping. Most of the increase in depth due to both breaches would probably be above the normal channel banks. However, the test flood breach causes a higher absolute maximum water surface elevation than the no-flood breach (EL 281.4 versus EL 281.2) and a greater increase in flow (170 cfs to 7,300 cfs = about 7,100 cfs increase versus 6,100 cfs increase for the no-flood breach). Therefore, the test flood breach causes more severe flooding and poses a greater hazard than the no-flood, sunny-day breach.

SECTION 6

EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The settlement of the crest relative to the core wall for both the dam and the dike indicates either that the foundation and embankment have compressed and/or that the embankment may have settled after construction when it became saturated. The fact that both shells have settled indicates that compression is the more likely cause of the observed displacements. However, the sinkhole-like feature just upstream of the core wall of the dike and the depression just upstream of the core wall of the dam both indicate that erosion under the core wall may be occurring intermittently.

6.2 Design and Construction Data

The design and construction data indicate that the core wall could crack due to differential settlement. Within the dam, about one-half of the length of the core wall is founded on piles and the other half is founded on foundation soil. The discontinuity of support leads to potential cracking.

For the dike the entire core is founded on soils of unknown thickness and type. Thus, longitudinal differential settlement of the core wall may be possible.

Also, the design data indicate that the left abutment of the dam and the right abutment of the dike (i.e., at the rock outcrop) are both quite steep. Such steep abutments lead to low stresses within the dam in adjacent zones, which in turn could become the focus of piping. There was no evidence seen that would indicate that such piping is now occurring.

6.3 Post-Construction Changes

In June 1942 a washout of the dike occurred. The washout was repaired by extending the core wall downward and replacing the downstream shell. It is not known what happened to the upstream shell or what repairs were made. Nor is the present depth of the core wall or the character of the downstream shell known. A sinkhole-like feature now exists upstream of the core wall in the dike, and the downstream crest has settled relative to the core wall. Thus, it appears that some changes have occurred since the washout was repaired. It is not known whether these

changes took place quickly and stopped, or whether they are occurring slowly. For these reasons, these observed features must be investigated if this dam is to remain in operation.

6.4 Seismic Stability

This dam is in Seismic Zone 2 and, in accordance with recommended guidelines, does not warrant seismic analysis.

SECTION 7

ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition

From a geotechnical standpoint, Norton Brook Dam is in POOR condition, primarily because of the presence of sinkhole-like features upstream from the core wall of both the dam and dike. Also, numerous large trees and brush cover all surfaces, a beaver pond obscures observation of any potential seeps downstream of the central part of the dam, and substantial settlement of the crest relative to the core wall has occurred.

From a hydraulic and hydrologic standpoint, the dam has adequate spillway capacity, because the test flood does not overtop the dam. In accordance with recommended guidelines established by the Corps of Engineers, the dam is classified as SMALL in size and as having a SIGNIFICANT hazard potential. Accordingly, a test flood equal to ONE-HALF PMF (probable maximum flood) was judged as appropriate within the recommended range of the 100-year flood to one-half PMF. The test flood does not overtop the dam, but results in a minimum freeboard of about 2.2 feet. Peak inflow for the test flood is 330 cfs. Peak outflow is reduced substantially by reservoir routing to 170 cfs. Total project discharge capacity at the top of the dam is due only to the drop inlet spillway (outlet works assumed closed) and is equal to 540 cfs, or 318% of the test-flood peak outflow.

b. Adequacy of Information

This Phase I Inspection was based primarily on the visual inspection and the hydraulic and hydrologic computations performed, coupled with sound engineering judgement. Available data consisted of USGS maps, the design/construction drawings, the record drawings of construction, and several inspection reports. The design calculations, construction specifications, data on the foundation and embankment soils, and operation and performance data were not available. The lack of such in-depth engineering data does not permit a comprehensive review. Therefore, the adequacy of this dam could not be assessed with respect to reviewing design, construction, and operation data.

c. Urgency

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the recommendations given in Section 7.2 and the remedial measures given in Section 7.3.

7.2 Recommendations

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should engage a registered engineer qualified in the design of earth dams to do the following work:

a. Investigate the cause of the sinkhole-like feature upstream from the core wall of the dike and the depression just upstream of the core wall of the dam, and make any necessary recommendations for repair.

b. Select proper material and procedures for backfilling the surfaces of the dam and dike after removal of the trees as recommended in Section 7.3.

c. Inspect the downstream face during and after the slopes have been cleared of trees and brush and the beaver pond has been removed, and make recommendations for monitoring the seeps or making other necessary repairs.

d. Make recommendations for repair of:

   1) the training walls of the outlet structure.
   2) the cracks in the sides of the spillway outlet conduit.
   3) the repair of the concrete damage on the intermediate pier legs of the service bridge.

e. Investigate the crack across the end of the service bridge at its abutment on the dam crest. Make recommendations for its modification as a true expansion joint if that is in fact its function as suspected.

7.3 Remedial Measures

a. Operation and Maintenance Procedures

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the following operation and maintenance procedures:

1) Cut all trees and brush from all surfaces of the dam and dike to a distance of about 20 feet downstream from the toeline. Remove roots of trees and place a properly selected and compacted soil material in the holes. Reseed with grass.

2) Keep surfaces of dam and dike mowed.

3) Remove beaver dam downstream from dam so that the pond behind it will be drained away from the toe of the dam.

4) Engage a qualified registered engineer to inspect the dam annually and report his findings to the Owner in writing.

5) Repair vandalism to outlet control tower.

6) Verify operating condition of all water intake valves and low level drain valve.

7) Remove rust and paint steel supports and railings on service bridge and piping in valve chamber.

8) Develop effective routine operation and maintenance procedures.

9) Develop a monitoring and warning system with an emergency action plan to insure proper and timely action during critical periods.

7.4 Alternatives

The water level in the reservoir may be lowered sufficiently so that the flood wave subsequent to any potential failure will not cause loss of downstream life or property.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST
DAM INSPECTION

DAM	Norton Brook Dam	DATE	October 24, 1979
ID NO.	VT 102	TIME	0800-1200
TOWN	Bristol	WEATHER	Drizzle, overcast, 55° F
COUNTY	Addison	W.S. ELEV.	381.2 UPSTREAM
STATE	Vermont		355 DOWNSTREAM

INSPECTION PARTY	RECORDER (X)
1. Kenneth Male, Gordon E. Ainsworth & Associates, Inc.	
2. Thomas Bennedum, Gordon E. Ainsworth & Associates, Inc.	X
3. John Kenworthy, Gordon E. Ainsworth & Associates, Inc.	
4. Steve Poulos, Geotechnical Engineers, Inc.	X
5. Kenneth Thiess, City Manager, City of Vergennes	
6. Carroll O'Connor, Supervisor of Public Works	
7. City of Vergennes	
8.	
9.	
10.	

PROJECT FEATURE/DISCIPLINE	INSPECTOR	REMARKS
1. H & H	T. Bennedum	-
2. Geotechnical	S. Poulos	Dam & Dike
3. Structural	T. Bennedum	-
4. Mechanical	T. Bennedum	-
5. Electrical	None	Not Applicable
6.		

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam DATE Oct. 24, 1979

PROJECT FEATURE - NAME -

DISCIPLINE Geotechnical NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	DAM EMBANKMENT - MAIN DAM TO RIGHT OF ROCK OUTCROP. SEE "DIKE EMBANKMENT" FOR PORTION TO THE LEFT OF ROCK OUTCROP.	
	Crest Elevation	EL 385
	Current Pool Elevation	EL 381.2
	Maximum Impoundment to Date	Unknown.
EI	Surface Cracks	Not observable.
GEI	Pavement Condition	None.
EI	Movement or Settlement of Crest	0+50L to 1+30L downstream and upstream shells have settled 1 ft relative to corewall. Crest narrow - forms peak over corewall. At 1+50R there is a depression upstream of core 1 to 1.5 ft deep and 4 ft dia.
EI	Lateral Movement	Not observable.
GEI	Vertical Alignment	Not observable.
EI	Horizontal Alignment	Not observable.
GEI	Condition at Abutment and at Concrete Structures	About 30 ft downstream from toe at right abutment is a wet, soft zone from about Sta 2+50R to 1+15R.
GEI	Indications of Movement of Structural Items on Slopes	Wing walls of outlet channel are tipped and cracked, probably due to frost action.
GEI	Trespassing on Slopes	Free access. Grassed path on crest. 3 or 4 beaver paths transverse to centerline from reservoir to pond downstream from toe. Beaver paths are eroded to dirt. One 6-in. animal hole on downstream slope.
GEI	Sloughing or Erosion of Slopes or Abutments	See "Movement or Settlement of Crest."
GEI	Rock Slope Protection - Riprap Failures	Riprap appears to be present but so overgrown that it is difficult to see.
EI	Unusual Movement or Cracking at or Near Toe	None observed.
GEI	Unusual Embankment or Downstream Seepage	Wet and soft along toe between right abutment and outlet structure. Sta 1+15L, seep running clear at ½ gpm

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam DATE Oct. 24, 1979

PROJECT FEATURE - NAME -

DISCIPLINE Geotechnical NAME S. J. Poulos

	AREA EVALUATED	CONDITION
EI	Unusual Embankment or Downstream Seepage (continued)	at left abutment contact about 3 ft elevation above toe. Seep running clear at approximately 2 gpm, 8 ft right of above seep. Rusty stains below both. Sta 1+00L at downstream toe up to 3 ft high there is a mushy zone about 15 ft by 15 ft.
EI	Piping or Boils	None observed.
EI	Foundation Drainage Features	None.
GEI	Toe Drains	None.
EI	Instrumentation System	None.
GEI	Vegetation	Grass and planted white pines. Generally 10-in.-dia. on downstream slope and 6-in.-dia. on crest and upstream slope.

3.

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam DATE Oct. 24, 1979

PROJECT FEATURE - NAME -

DISCIPLINE Geotechnical NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	DIKE EMBANKMENT - DIKE IS TO LEFT OR ROCK OUTCROP THAT FORMS LEFT ABUTMENT OF DAM.	
	Crest Elevation	EL 385
	Current Pool Elevation	EL 381.2
	Maximum Impoundment to Date	Unknown
GEI	Surface Cracks	None observable.
GEI	Pavement Condition	None.
GEI	Movement or Settlement of Crest	Sta 2+80L: 15 ft wide subsidence, up to 3 ft deep at center, upstream of corewall. Looks like sinkhole. Sta 1+60L to 2+65L downstream shell has subsided 1 to 2 ft below top of corewall. Cover over corewall is about 6".
GEI	Lateral Movement	Not observable.
GEI	Vertical Alignment	Not observable.
GEI	Horizontal Alignment	Not observable.
GEI	Condition at Abutment and at Concrete Structures	Abutments appear satisfactory.
GEI	Indications of Movement of Structural Items on Slopes	No structural items.
GEI	Trespassing on Slopes	Free access. No animal holes found.
GEI	Sloughing or Erosion of Slopes or Abutments	See "Movement or Settlement of Crest."
GEI	Rock Slope Protection - Riprap Failures	Riprap not visible at Sta 2+80L to 2+10L. Remainder is overgrown but appears in satisfactory condition.
GEI	Unusual Movement or Cracking at or Near Toes	Not observable. Overgrown.
GEI	Unusual Embankment or Downstream Seepage	Not observable. Overgrown.
GEI	Piping or Boils	Not observable. Overgrown.
GEI	Foundation Drainage Features	None.
GEI	Toe Drains	None.
GEI	Instrumentation System	None.
GEI	Vegetation	White pine up to 2-ft-dia.

VISUAL INSPECTION CHECKLIST

PROJECT: Norton Brook Dam

DATE: Oct. 24, 1979

PROJECT FEATURE: Structural/H & H

NAME: T. Bennedum

DISCIPLINE: Geotechnical

NAME: S. J. Poulos

	AREA EVALUATED	CONDITION
	<u>OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE</u>	
	a. Approach Channel	
G I	Slope Conditions	N.A.
GEI	Bottom Conditions	Not observable. In reservoir.
C I	Rock Slides or Falls	None.
	Log Boom	None.
	Debris	None.
	Condition of Concrete Lining	Not observable. In reservoir.
GEI	Drains or Weep Holes	N.A.
	b. Intake Structure	
	Condition of Concrete	Good
	Stop Logs and Slots	No stop logs. Visible portion of bar screen above water in fair condition. Slots are rusted.

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam DATE Oct. 24, 1979

PROJECT FEATURE Structural/Mechanical NAME T. Bennedum

DISCIPLINE Geotechnical NAME S. J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	
a. Concrete and Structural	Good. Brick work sound. Doors and windows broken. Vandalism evident.
General Condition	
Condition of Joints	Good.
Spalling	None observed.
Visible Reinforcing	None observed.
Rusting or Staining of Concrete	None observed.
Any Seepage or Efflorescence	Efflorescence at const. joint 12' $\pm$ above valve chamber floor. Worst on lake side. No actual seeps. Good.
Joint Alignment	
Unusual Seepage or Leaks in Gate Chamber	None observed.
Cracks	None observed.
Rusting or Corrosion of Steel	Some on ladder, piping & valves.
b. Mechanical and Electrical	
Air Vents	None
Float Wells	None
Crane Hoist	None
Elevator	None
Hydraulic System	None
Service Gates	3-8" valves w/valve stands & HW's. Low-5/8 open, middle -2/8 open, high-stand removed. Rusted. Fair.
Emergency Gates	One 14" gear valve w/HW. Operable condition unknown. Rusted. Fair.
Lightning Protection System	None
Emergency Power System	None
Wiring and Lighting System	None

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam

DATE Oct. 24, 1979

PROJECT FEATURE Structural/H & H

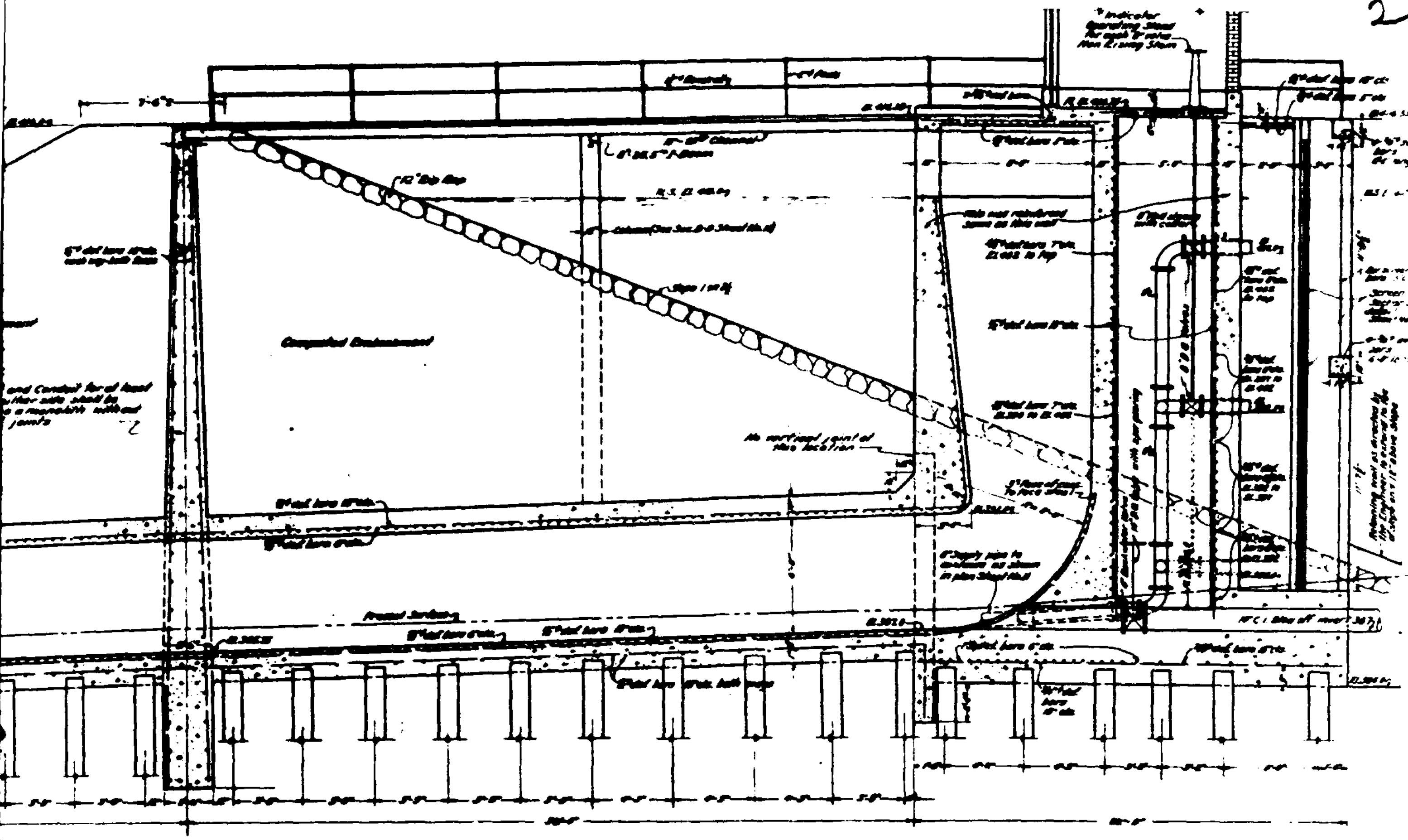
NAME T. Bennedum

DISCIPLINE Geotechnical

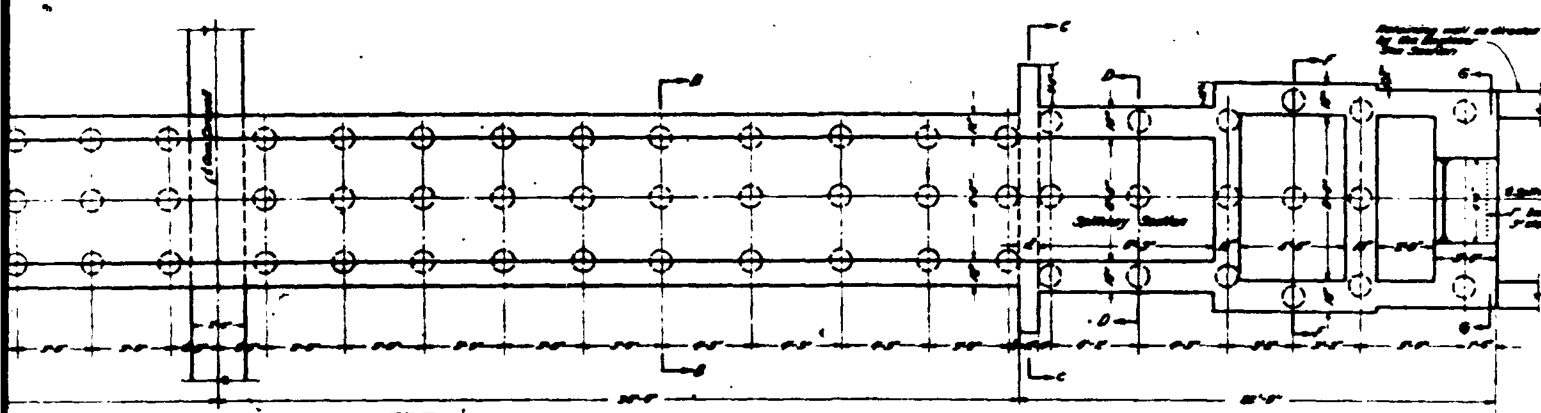
NAME S. J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	
General Condition of Concrete	Fair to good. Flowing water made inspection difficult. Only could get to within 10' ± of transition.
Rust or Staining on Concrete	None observed.
Spalling	None observed.
Erosion or Cavitation	None observed.
Cracking	About 26' upstream from end of conduit, conduit is cracked along bottom and up sides. Apparent settlement. Minor seepage at crack. Some evidence of repair w/grout at at this crack and other joints.
Alignment of Monoliths -See A & B below.	
Alignment of Joints -See A & B below.	
Numbering of Monoliths - None.	
A.	At 4.5' ± long section through corewall, conduit has settled on either side, about 2.5" upstream and 1.5" downstream. Joints have been grouted all around, and are tight. At downstream joint, appear to be 2 grout pipes w/plugs set in invert, 1' ± either side of centerline.
B.	At next two joints downstream (11.5' ±) sections) there appears to be minor seepage and evidence of grout repair.

2



SPILLWAY SECTION



FOUNDATION & PILE PLAN

CITY OF VERGENNES

ADDISON COUNTY, VERMONT

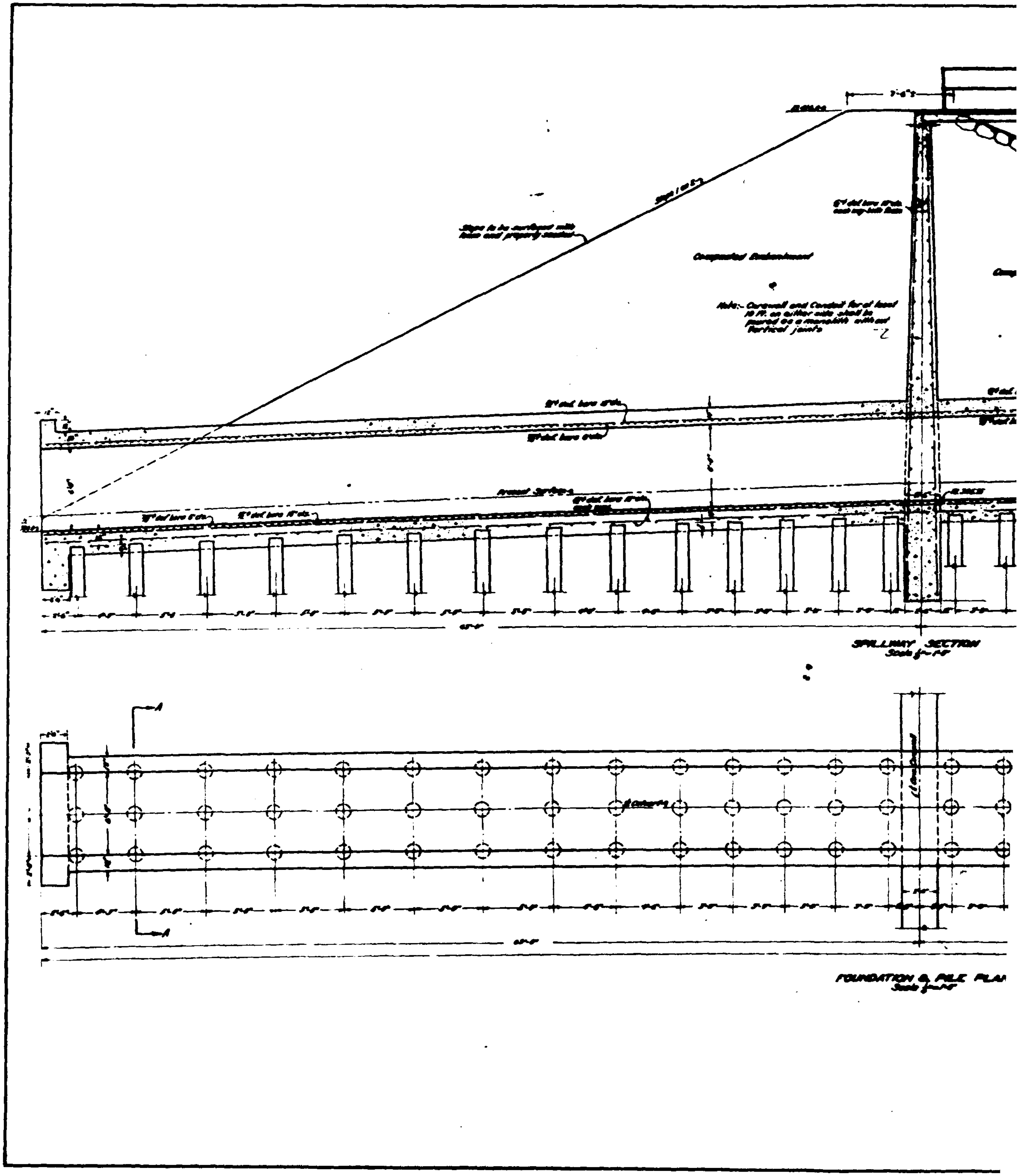
NEW GRAVITY WATER SUPPLY SYSTEM

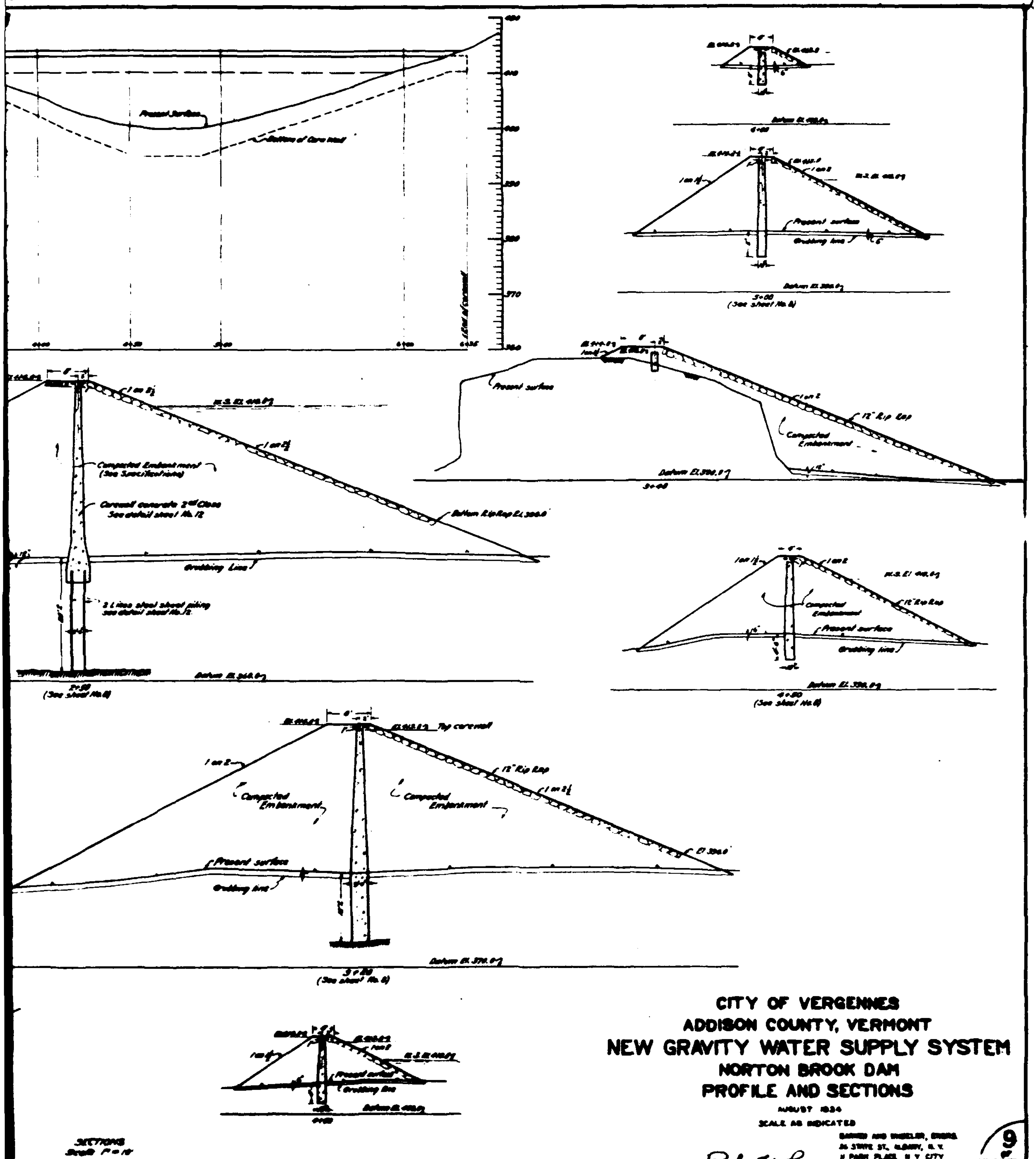
NORTON BROOK DAM AND RESERVOIR

DETAILS OF SPILLWAY

10 OF 17 SHEETS

TES INC 79-382


Compacted Embankment
SPILLWAY SECTION



CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
NORTON BROOK DAM
PROFILE AND SECTIONS

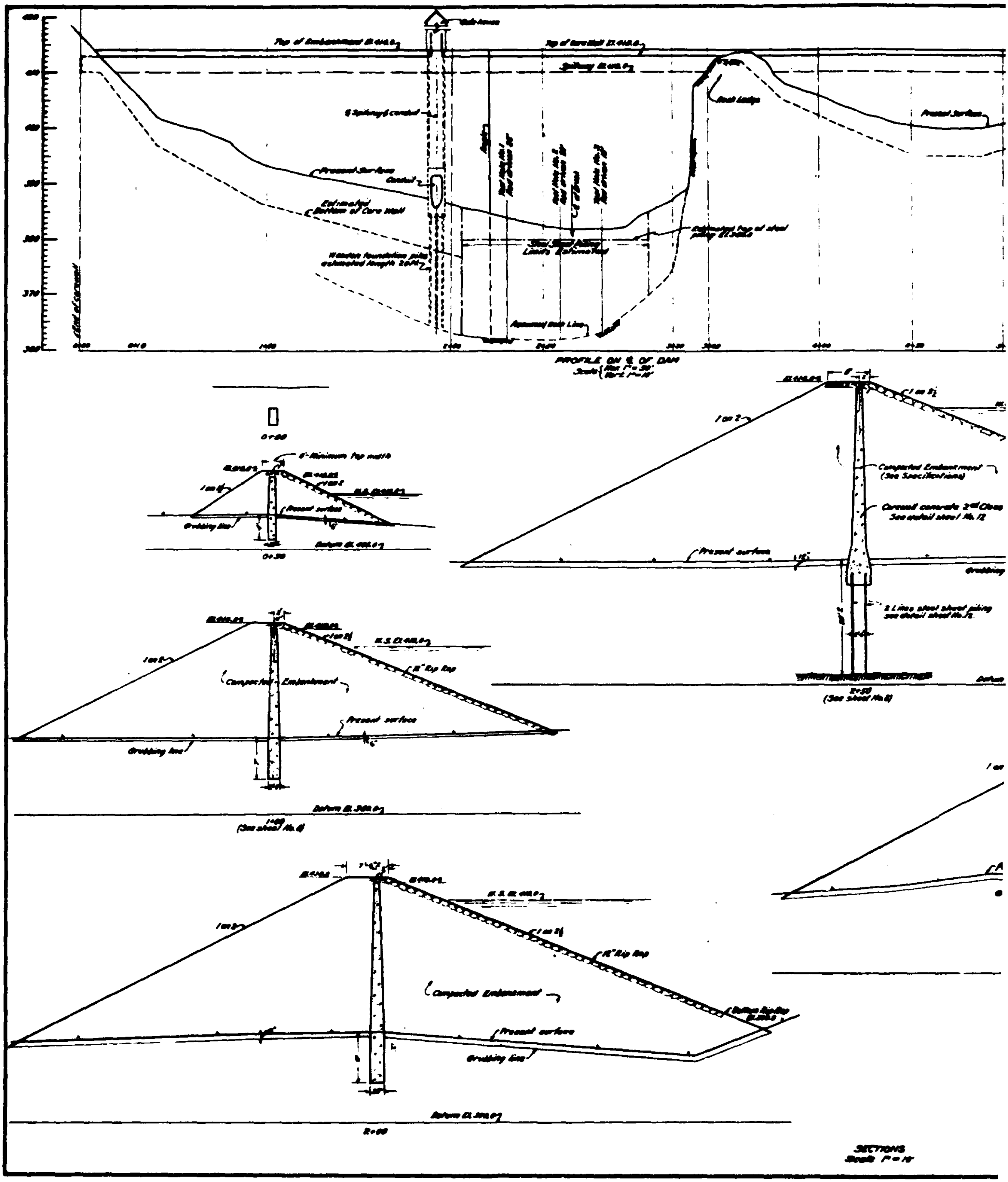
AUGUST 1934
SCALE AS INDICATED

BARNES AND WHEELER, ENGRS.
36 STATE ST., ALBANY, N.Y.
11 PARK PLACE, N.Y. CITY

9 OF 17 SHEETS

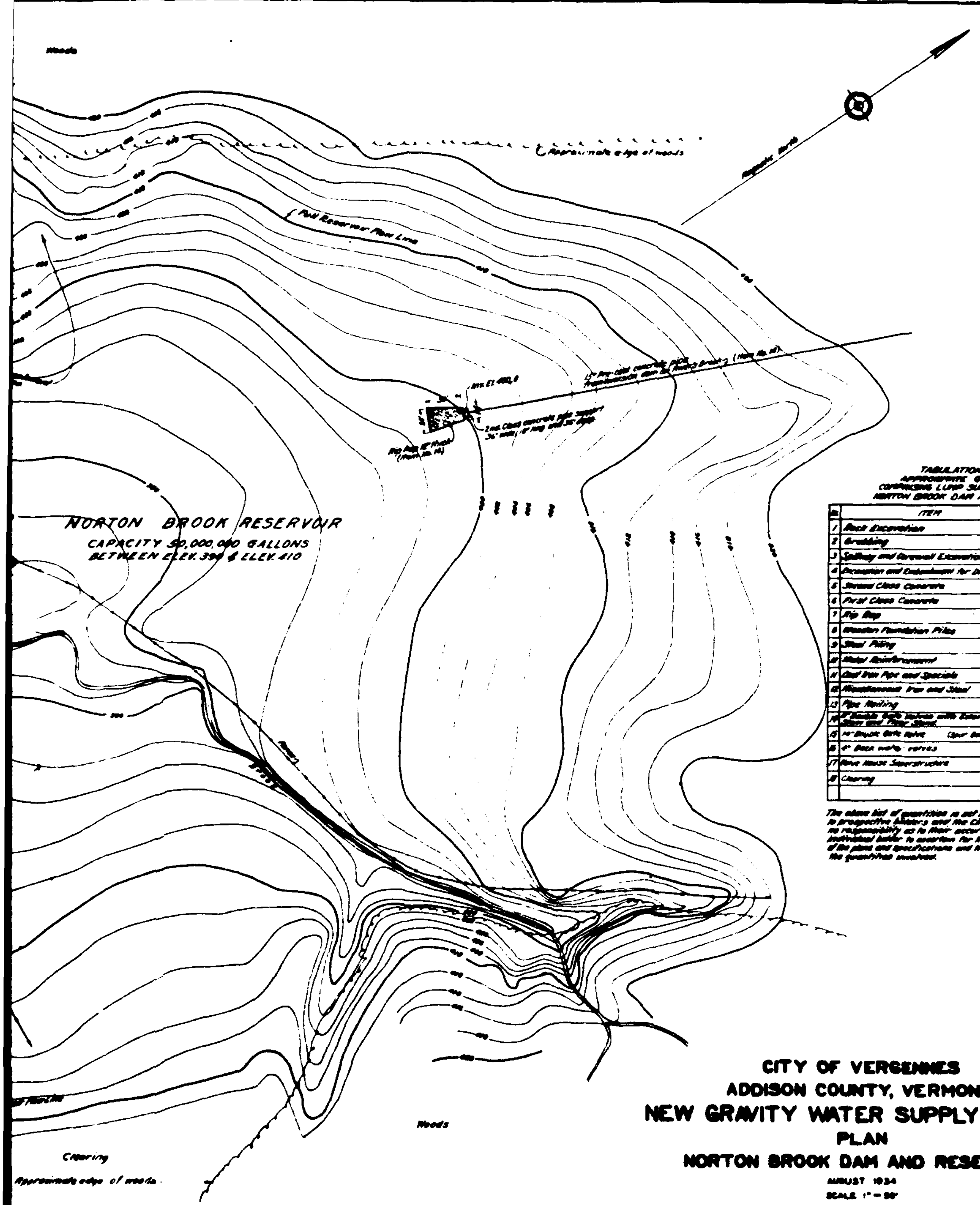
ES INC 79-382

B2-3



REDUCED TO 47% OF ORIGINAL

GORDON E. AINSWORTH & ASSOCIATES INC 79-382



TABULATIONS OF
APPROXIMATE QUANTITIES
COMPRISING LUMP SUM ITEM No. 13
NORTON BROOK DAM AND RESERVOIR

No.	ITEM	MEASURE	QUANT.
1	Rock Excavation	Cu Yd	100
2	Grubbing	"	1,350
3	Spillway and Corewall Excavation	"	500
4	Excavation and Embankment for Dam	"	25,300
5	Second Class Concrete	"	920
6	First Class Concrete		230
7	Rip Rap	"	1,200
8	Wooden Foundation Piles	Lin. Ft.	1,000
9	Steel Piling	Sq. Ft.	3,400
10	Metal Reinforcement	Lbs	50,000
11	Cast Iron Pipe and Specials	Tons	6
12	Miscellaneous Iron and Steel	[illegible]	4,200
13	Pipe Railing	[illegible]	110
14	Double Gate Valves with Extension Stem and Floor Stand	each	3
15	Double Gate Valve (Spur Gearing)	"	1
16	4" Back water valves	"	2
17	Valve House Superstructure	Lump Sum	1
18	Clearing	"	1

The above list of quantities is set forth merely as a guide to prospective bidders and the City of Vergennes assumes no responsibility as to their accuracy and expects each individual bidder to ascertain for himself by personal examination of the plans and specifications and the site of the work as to the quantities involved.

CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
PLAN
NORTON BROOK DAM AND RESERVOIR

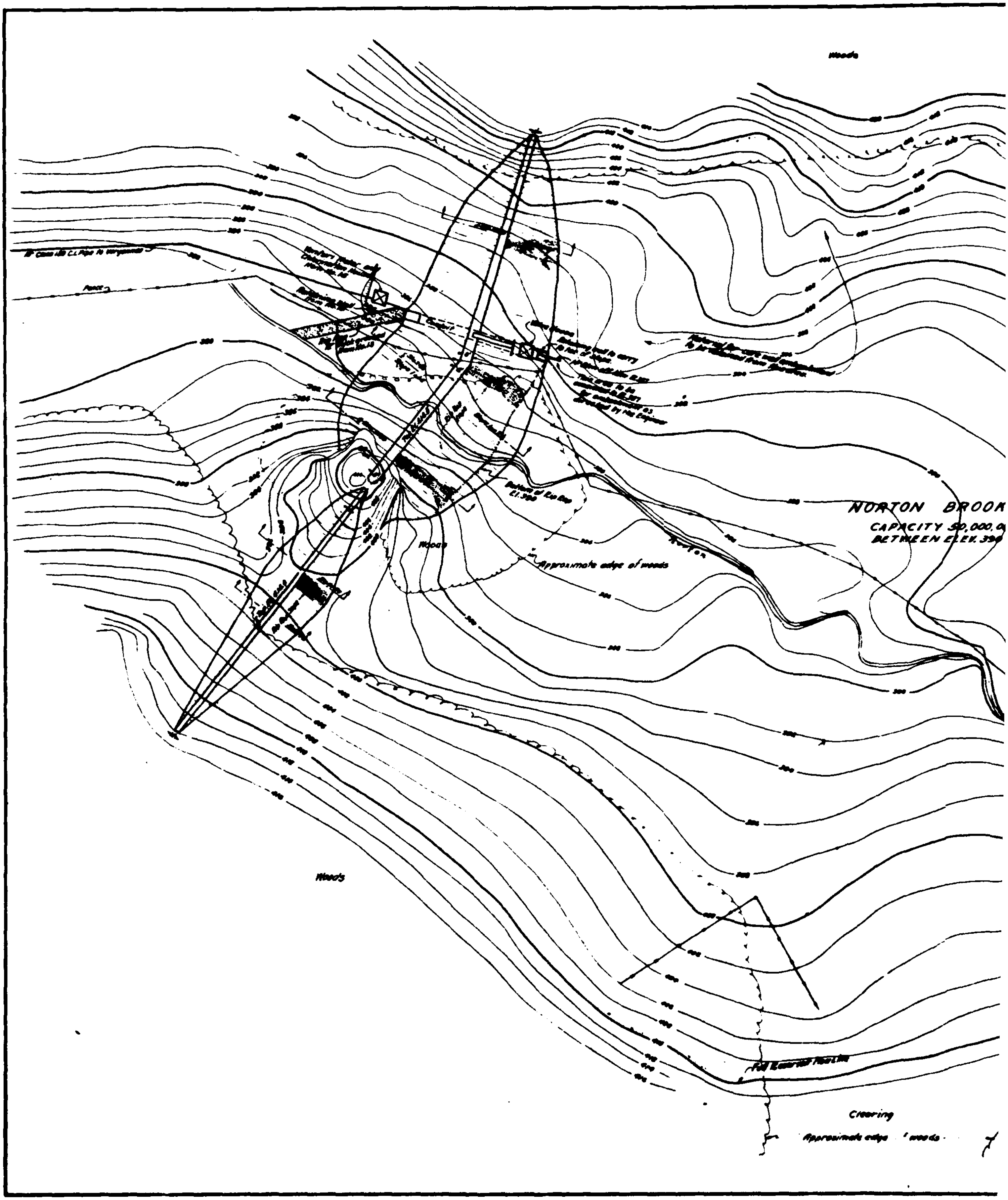
AUGUST 1934
SCALE 1" = 50'

BARKER AND WHEELER, ENGRS.
36 STATE ST., ALBANY, N. Y.
11 PARK PLACE, N. Y. CITY

8 OF 17 SHEETS

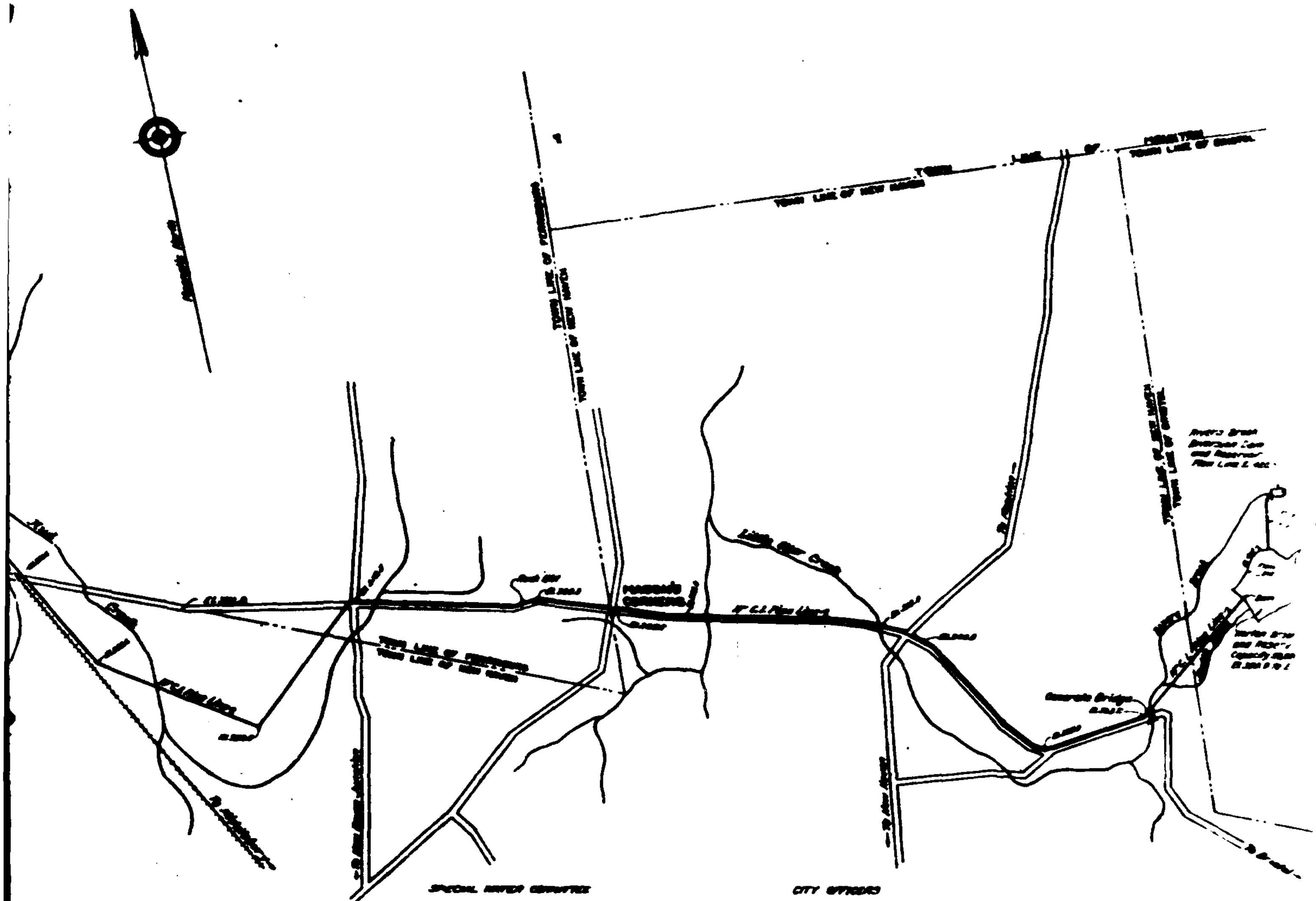
INC 79-382

B2-2



REDUCED TO 47% OF ORIGINAL

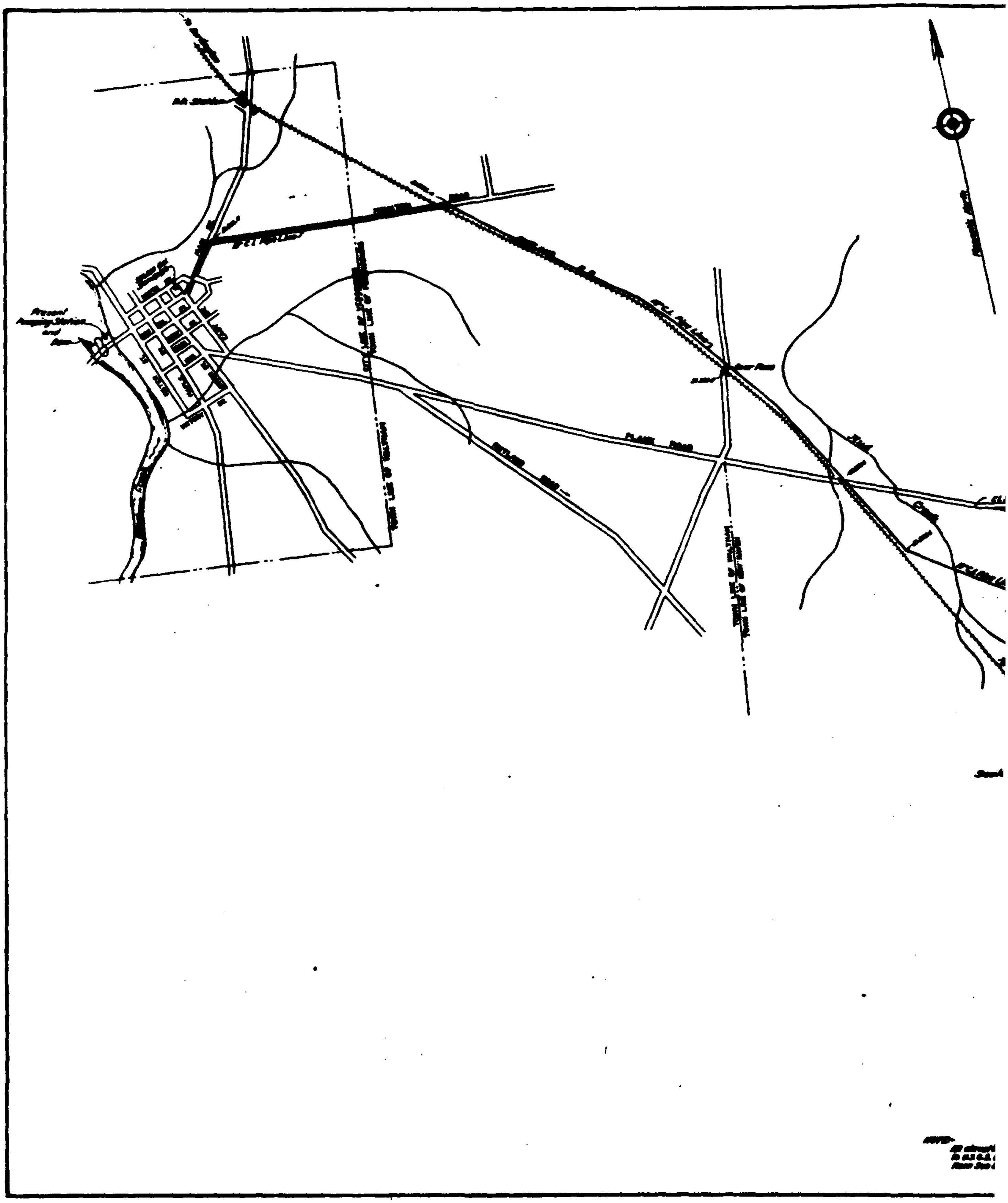
GORDON E AINSWORTH & ASSOCIATES INC 79-382



SPECIAL WATER COMMITTEE

CITY OFFICERS

CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
GENERAL LAYOUT



REDUCED TO 47 % OF ORIGINAL

GORDON E. AINSWORTH & ASSOCIATES INC. 79-382

APPENDIX B

Section B2

PLANS, SECTIONS, AND DETAILS

TABLE OF CONTENTS

	Page
DESIGN/CONSTRUCTION DRAWINGS[1] - August 1934	
General Layout - Sheet 1 of 17	B2-1
Plan - Sheet 8 of 17	B2-2
Profile and Sections - Sheet 9 of 17	B2-3
Details of Spillway - Sheet 10 of 17	B2-4
Miscellaneous Plans and Sections - Sheet 11 of 17	B2-5
Miscellaneous Details - Sheet 12 of 17	B2-6
General Layout, Profile and Sections - Sheet 13 of 17	B2-7
Rivers Brook Diversion Dam, Plans and Sections - Sheet 14 of 17	B2-8
REVISED DESIGN/CONSTRUCTION DETAILS - February 1935	
Spillway Foundation - Sheet A	B2-9
Foundation for Corewall - Sheet B	B2-10
Foundation for Corewall - Sheet B - revised May 6, 1935	B2-11
Spillway and Conduit Construction - Sheet C - May 1935, revised 5/6/59	B2-12
RECORD DRAWINGS OF CONSTRUCTION - Sheet 3 of 3 - January 1936	B2-13
DRAWING OF DAM SEEPAGE LOCATION - July 1936	B2-14

(1) Original set consists of 17 drawings covering the entire water supply system. Only those drawings pertinent to the dam are included herein.

APPENDIX B

SECTION B1

LISTING OF LOCATIONS FOR AVAILABLE RECORDS AND DATA

a) Owner: City of Vergennes
P.O. Box 169
Vergennes, Vermont 05491
Attention: Kenneth C. Thiess
City Manager
(802) 877-3637

1) plans, sections, and details
2) construction bid summary sheet
3) record drawings
4) correspondence

b) Designer: Barker & Wheeler Engineers
36 State Street
Albany, New York
(This firm is no longer in business)

c) Construction Contractor: W. G. Fritz Co.
69 Main Street
West Orange, New Jersey
(201) 731-0572
(per Tel. Co. information)
(Unable to confirm business status and phone number).

d) Agency of Environmental Conservation
Department of Water Resources
Water Quality Division
Montpelier, Vermont 05602
Attention: A. Peter Barranco, Jr., P.E.,
Dam Safety Engineer
(802) 828-2761

1) correspondence
2) inspection reports
3) some plans

APPENDIX B

ENGINEERING DATA

Section	Description
B1	Listing of Locations for Available Records and Data
B2	Plans, Sections, and Details
B3	Copies of Past Inspection Reports and Data

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam DATE Oct. 24, 1979

PROJECT FEATURE Structural NAME T. Bennedum

DISCIPLINE Geotechnical NAME S. J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	Bridge appears bolted to spillway structure and cast into or on corewall.
a. Super Structure	
Bearings	Not observable on pier. Suspect none.
Anchor Bolts	Bolts to spillway structure good condition.
Bridge Seat	Not observable at pier. N.A. at abutments.
Longitudinal Members	Good. Some rust on steel.
Underside of Deck	Good.
Secondary Bracing	Fair. Steel rusted.
Deck	Concrete deck good condition.
Drainage System	None. Drains over sides.
Railings	Sound steel pipe railings. Paint flaked.
Expansion Joints	None.
Paint	All steel needs new paint.
b. Abutment & Piers	
General Condition of Concrete	Ice damage to pier concrete at waterline. Concrete in abutments good.
Alignment of Abutment	Good.
Approach to Bridge	Level from dam crest.
Condition of Seat & Backwall	Crack at abutment on dam crest. Spillway structure abutment good condition.

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam DATE Oct. 24, 1979

PROJECT FEATURE Structural/ H & H NAME T. Bennedum

DISCIPLINE Geotechnical NAME S. J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	N.A. Drop inlet spillway.
General Condition	N.A.
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	N.A.
Floor of Approach Channel	Not observable
b. Weir and Training Walls	Hard to inspect due to configuration and flowing water.
General Condition of Concrete	Good
Rust or Staining	None observed.
Spalling	Minor spalling at inside corners of corner posts.
Any Visible Reinforcing	None observed.
Any Seepage or Efflorescence	None observed.
Drain Holes	Same as outlet channel. See page A-8 .
c. Discharge Channel	
General Condition	
Loose Rock Overhanging Channel	
Trees Overhanging Channel	
Floor of Channel	
Other Obstructions	

VISUAL INSPECTION CHECKLIST

PROJECT Norton Brook Dam

DATE Oct. 24, 1979

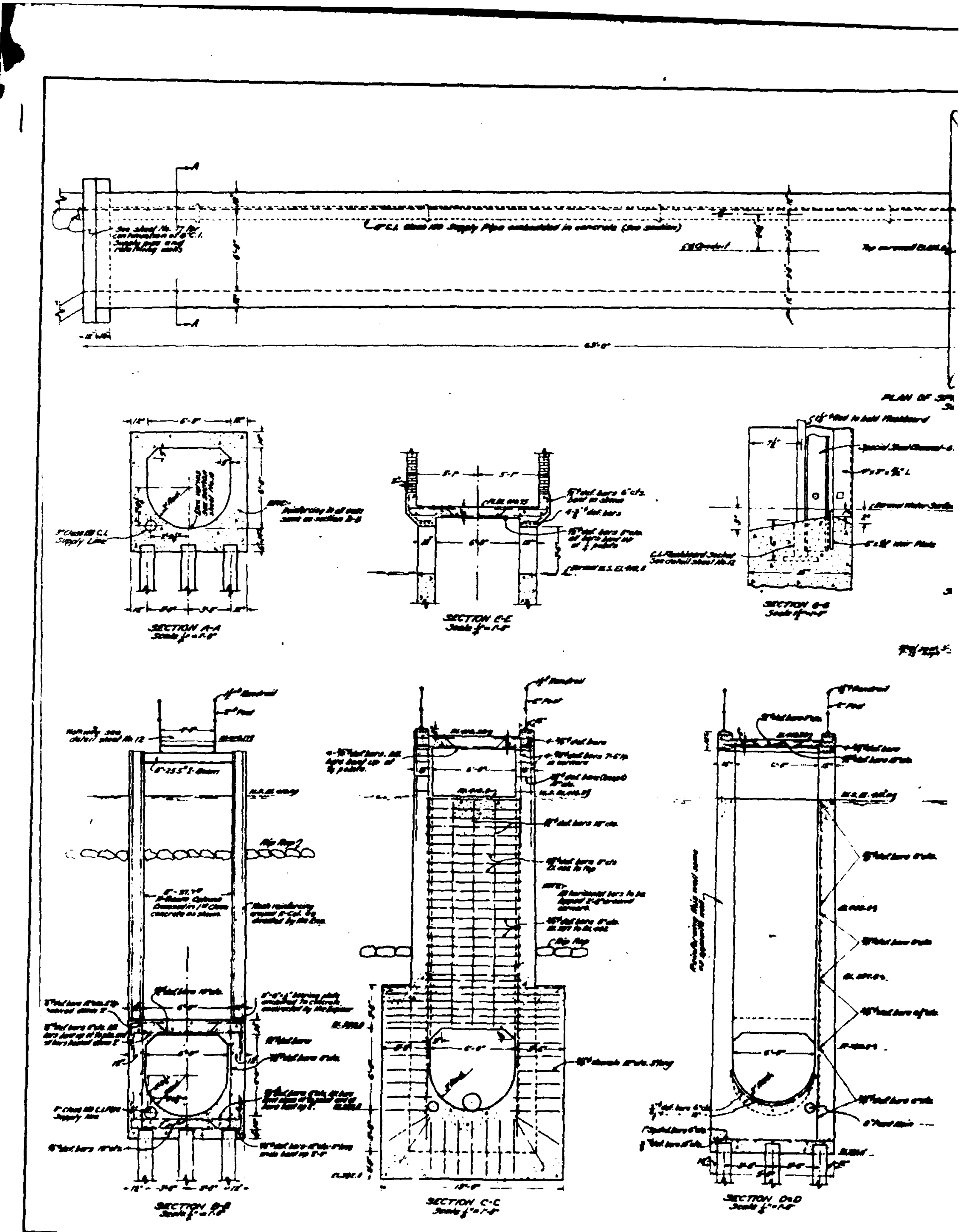
PROJECT FEATURE Structural/ H & H

NAME T. Bennedum

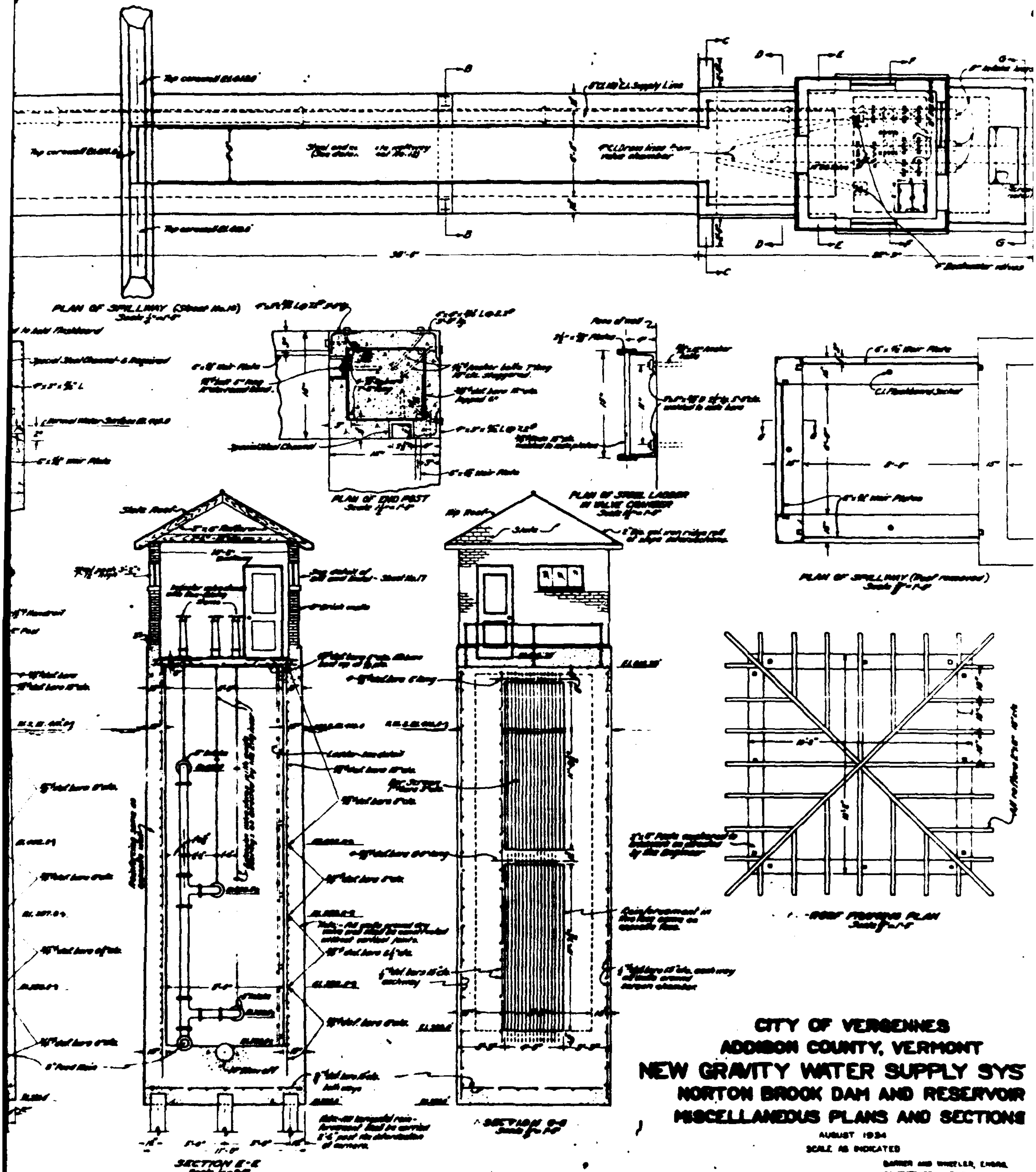
DISCIPLINE Geotechnical

NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	Remains of old wooden door over end of conduit evident.
	General Condition of Concrete	Fair.
	Rust or Staining	Some rust or staining w/moss growth.
	Spalling	Spalling and cracking at angle point joints w/training walls. Minor erosion in invert.
	Erosion or Cavitation	
	Visible Reinforcing	Rebar visible at angle point joint w/right training wall.
	Any Seepage or Efflorescence	None observed.
	Condition at Joints	Good except at angle point joints w/training walls.
GEI	Drain holes	None.
(I	Channel	
GFI	Loose Rock or Trees Overhanging Channel	No rock. Trees abundant.
GEI	Condition of Discharge Channel	Fair condition. Wing walls are cracked and tilted inward probably due to frost action. Channel is fully vegetated. Outlet runs into beaver pond downstream.



REDUCED TO 47% OF ORIGINAL

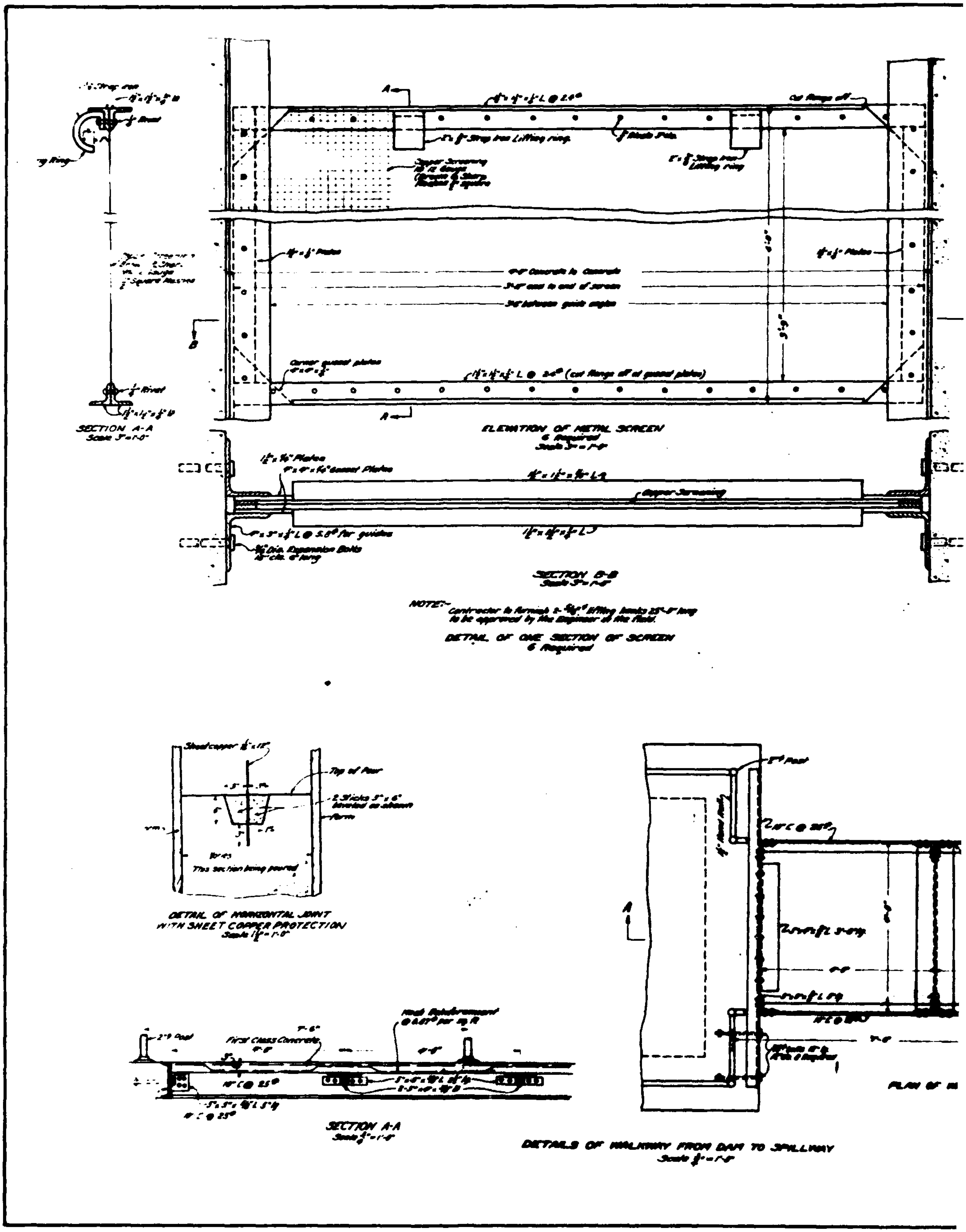


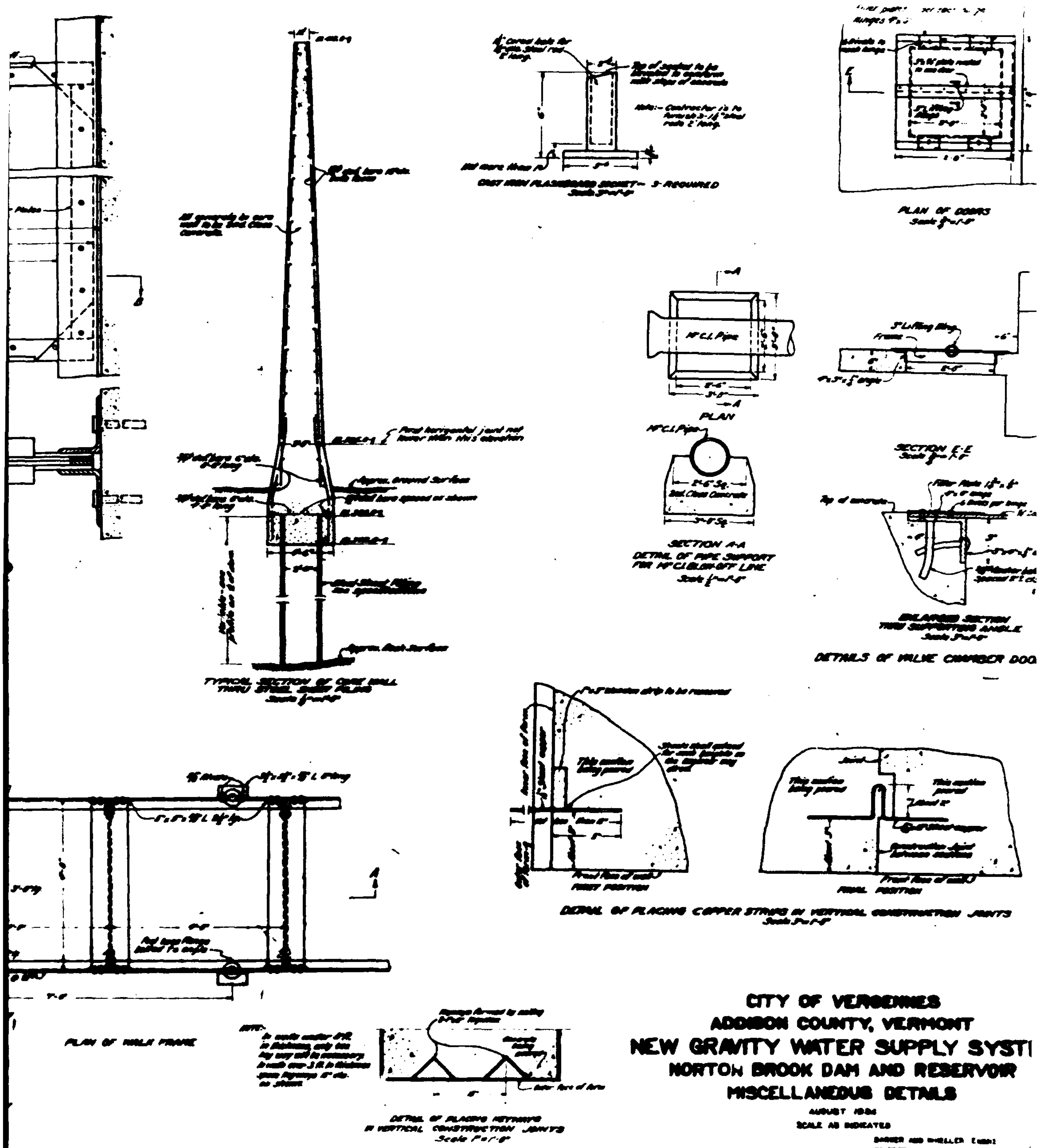
CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYS
NORTON BROOK DAM AND RESERVOIR
MISCELLANEOUS PLANS AND SECTIONS
AUGUST 1934
SCALE AS INDICATED

BARKER AND WHEELER, ENGRS.
36 STATE ST, ALBANY, N.Y.
11 PARK PLACE, N.Y. CITY

INC 79-382

B2-5


SECTION A-A
ELEVATION OF METAL SCREEN
6 Required
SECTION B-B
Copper Screening
Corner gusset plates
NOTE:-
DETAIL OF ONE SECTION OF SCREEN
6 Required
Top of Pour
This section being poured
DETAIL OF HORIZONTAL JOINT
WITH SHEET COPPER PROTECTION
First Class Concrete
SECTION A-A
DETAILS OF WALKWAY FROM DAM TO SPILLWAY



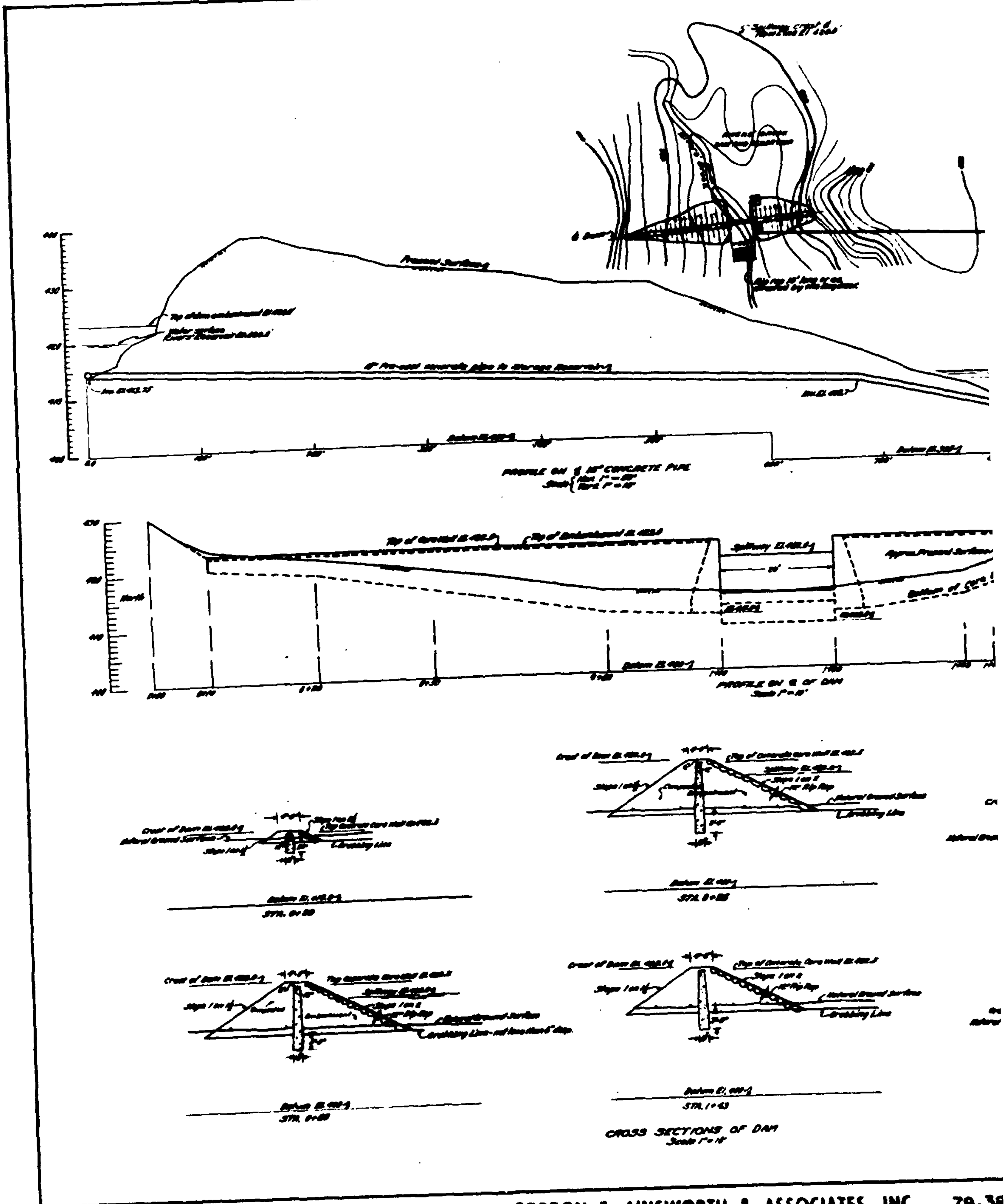
CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYST
NORTON BROOK DAM AND RESERVOIR
MISCELLANEOUS DETAILS

AUGUST 1934

SCALE AS INDICATED

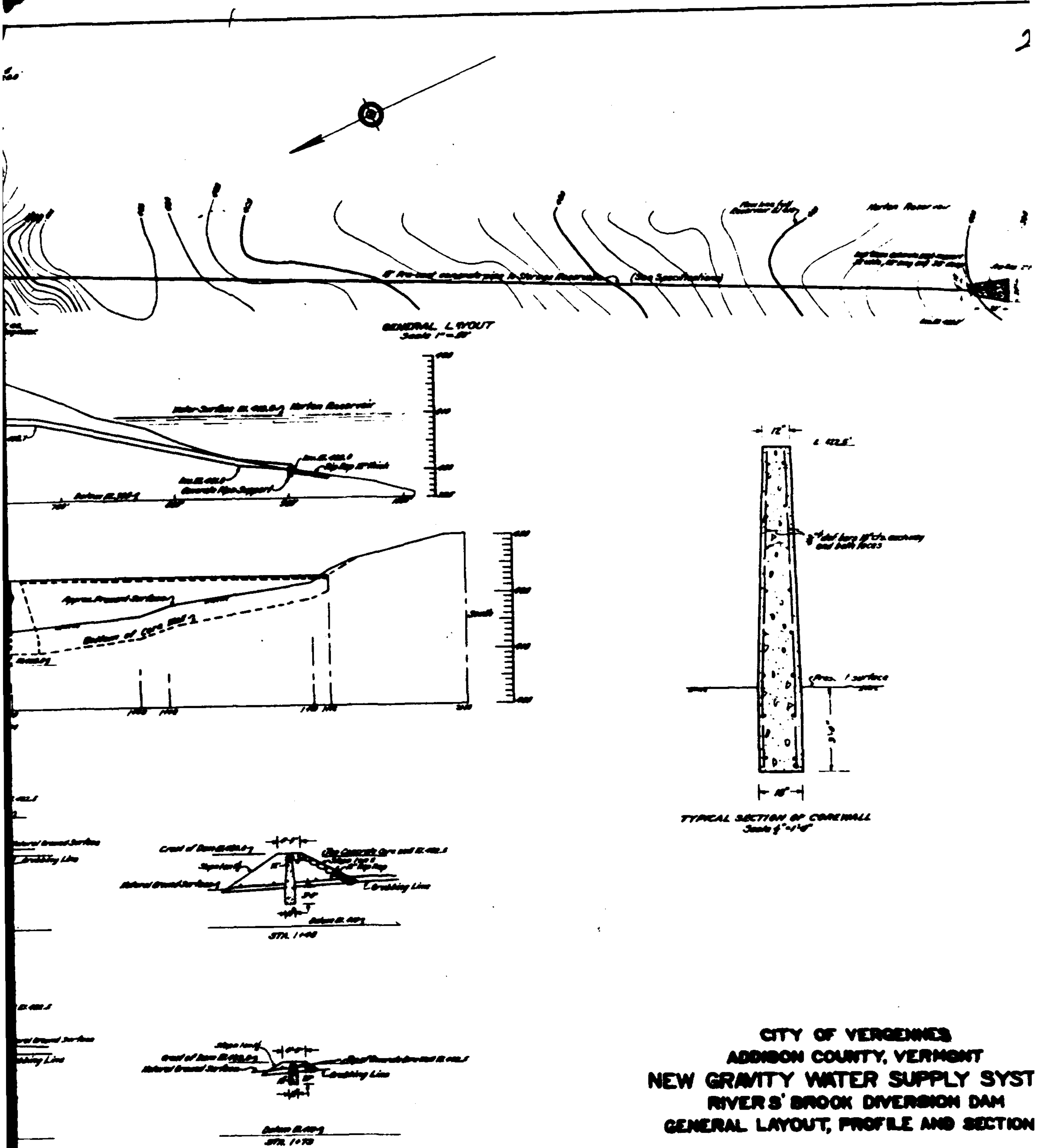
INC 79-382

B2-6



GORDON E. AINSWORTH & ASSOCIATES INC. 79-38

REDUCED TO 47% OF ORIGINAL



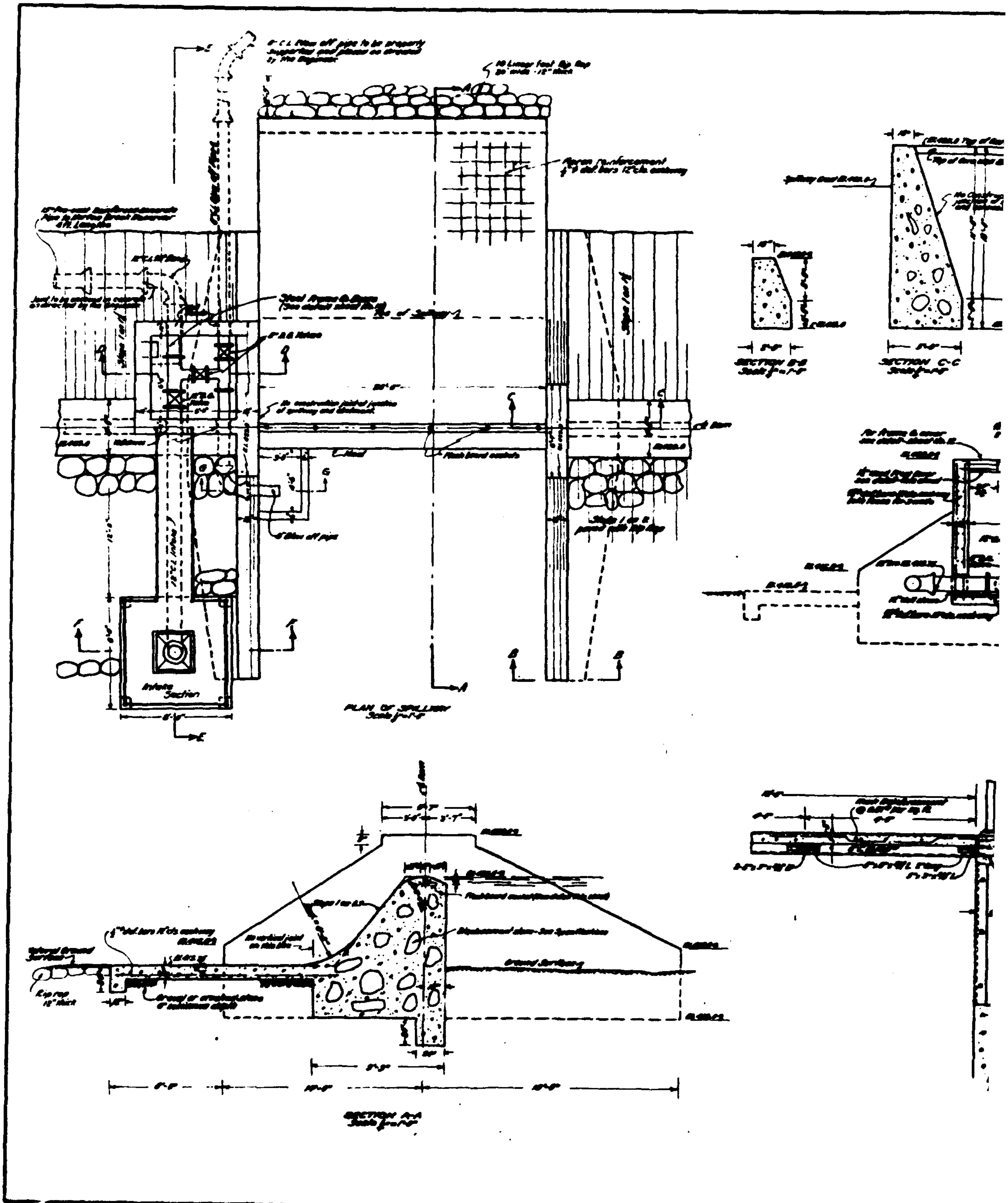
CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYST
RIVERS' BROOK DIVERSION DAM
GENERAL LAYOUT, PROFILE AND SECTION

AUGUST 1924
SCALE AS INDICATED

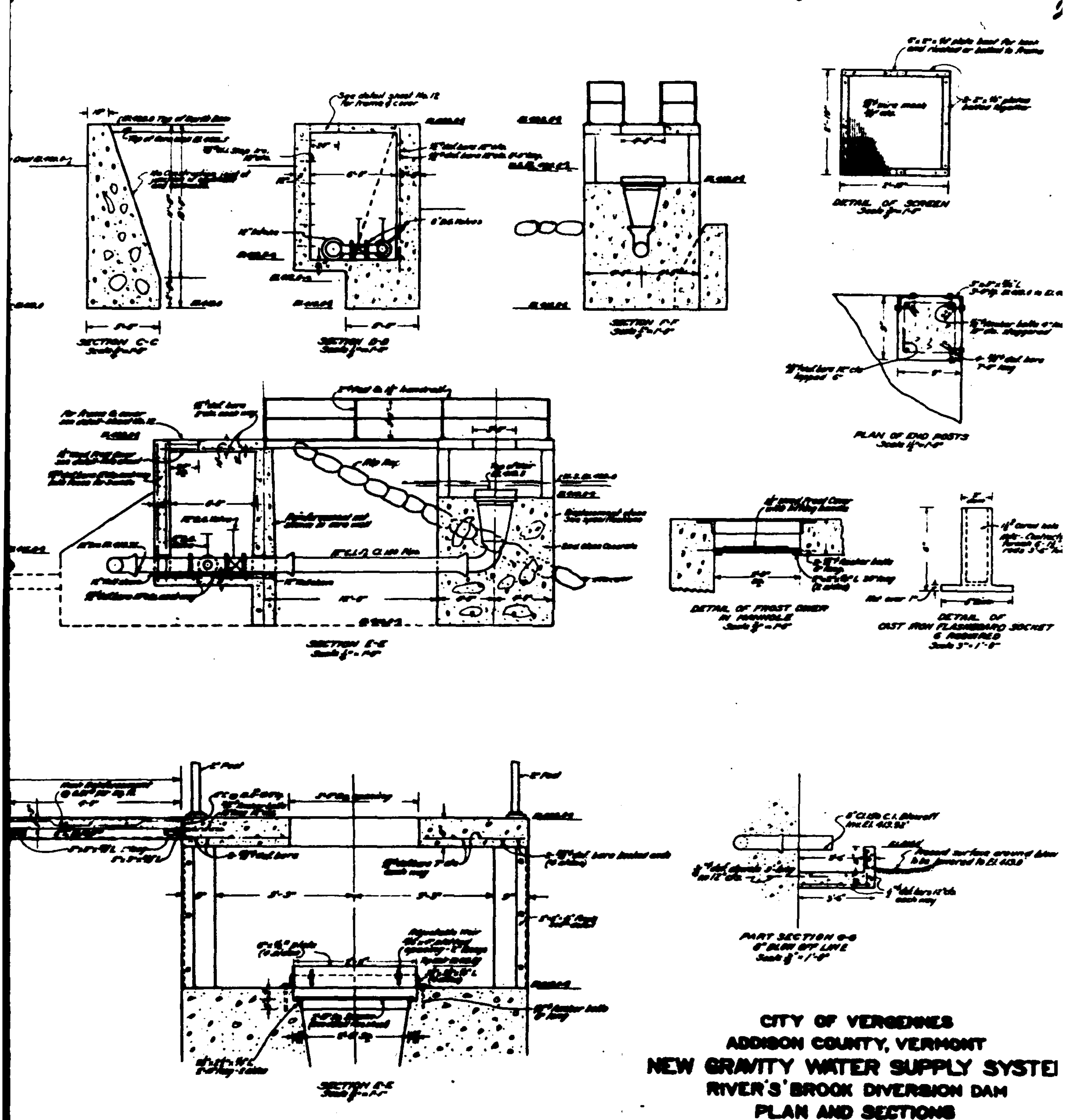
BARNES AND WHEELER, ENGRS
30 STATE ST. ALBANY, N.Y.

B2-7

ES INC 79-382



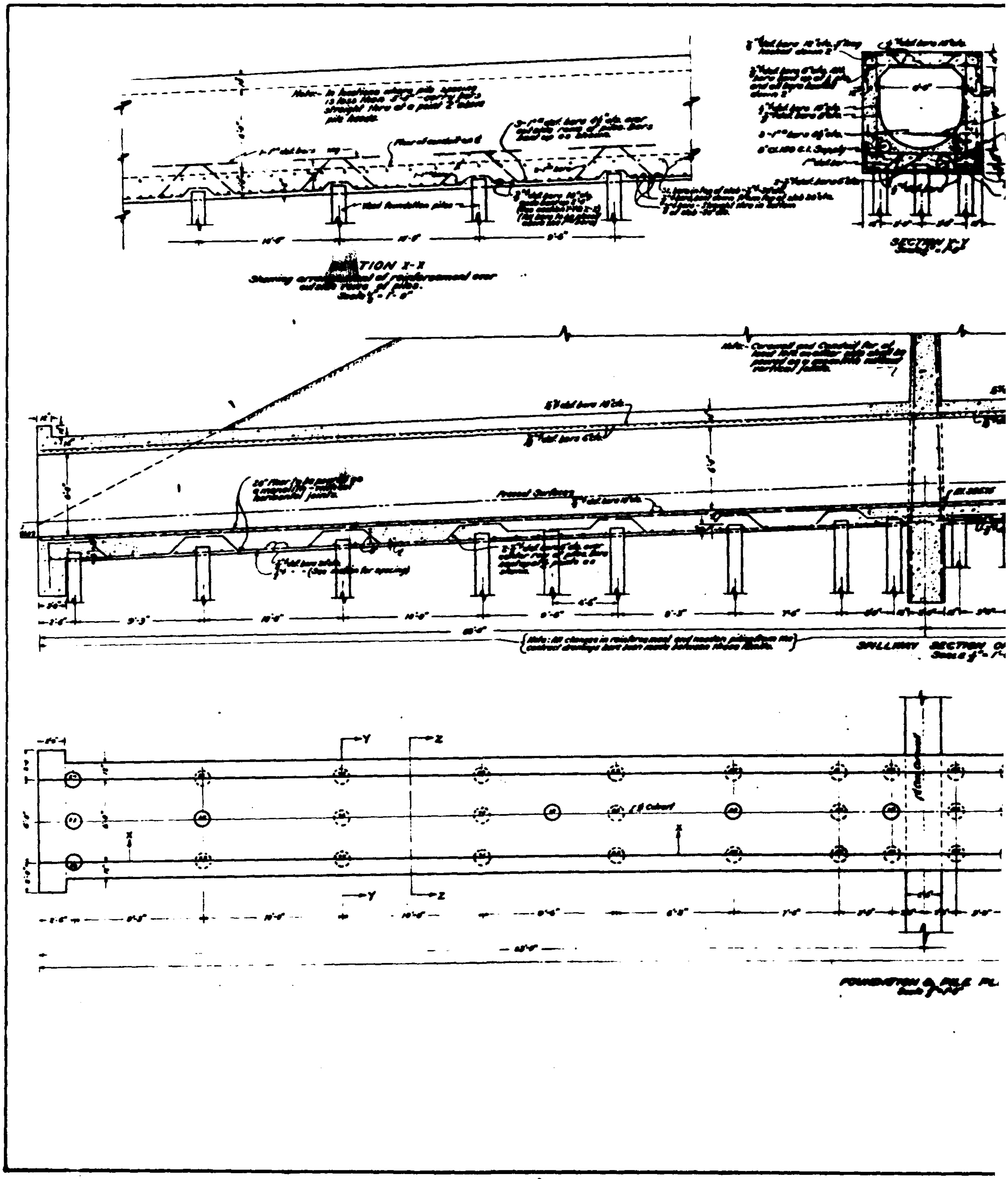
REDUCED TO 47% OF ORIGINAL



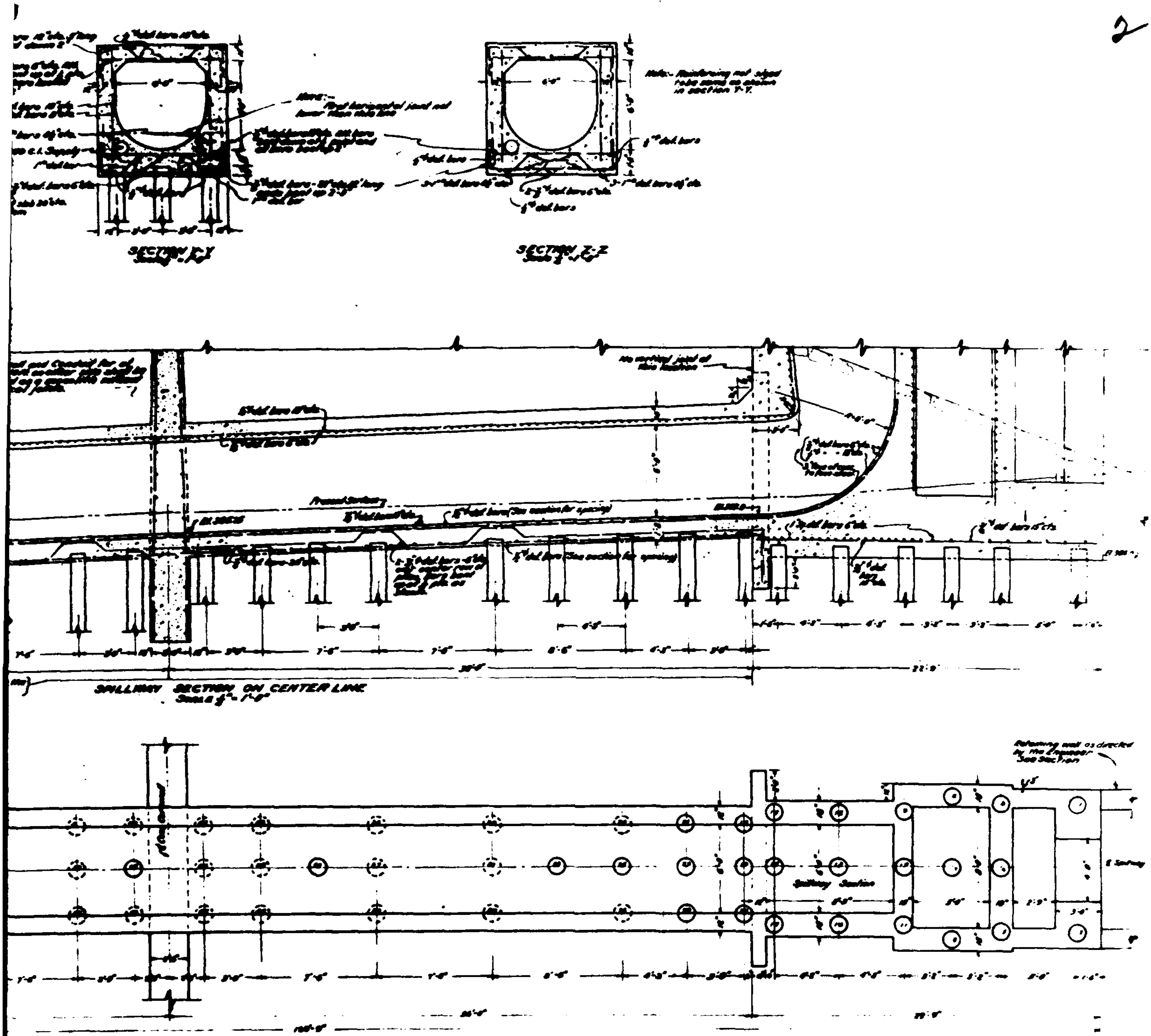
CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTE
RIVER'S BROOK DIVERSION DAM
PLAN AND SECTIONS

TES INC 79-382

B2-8



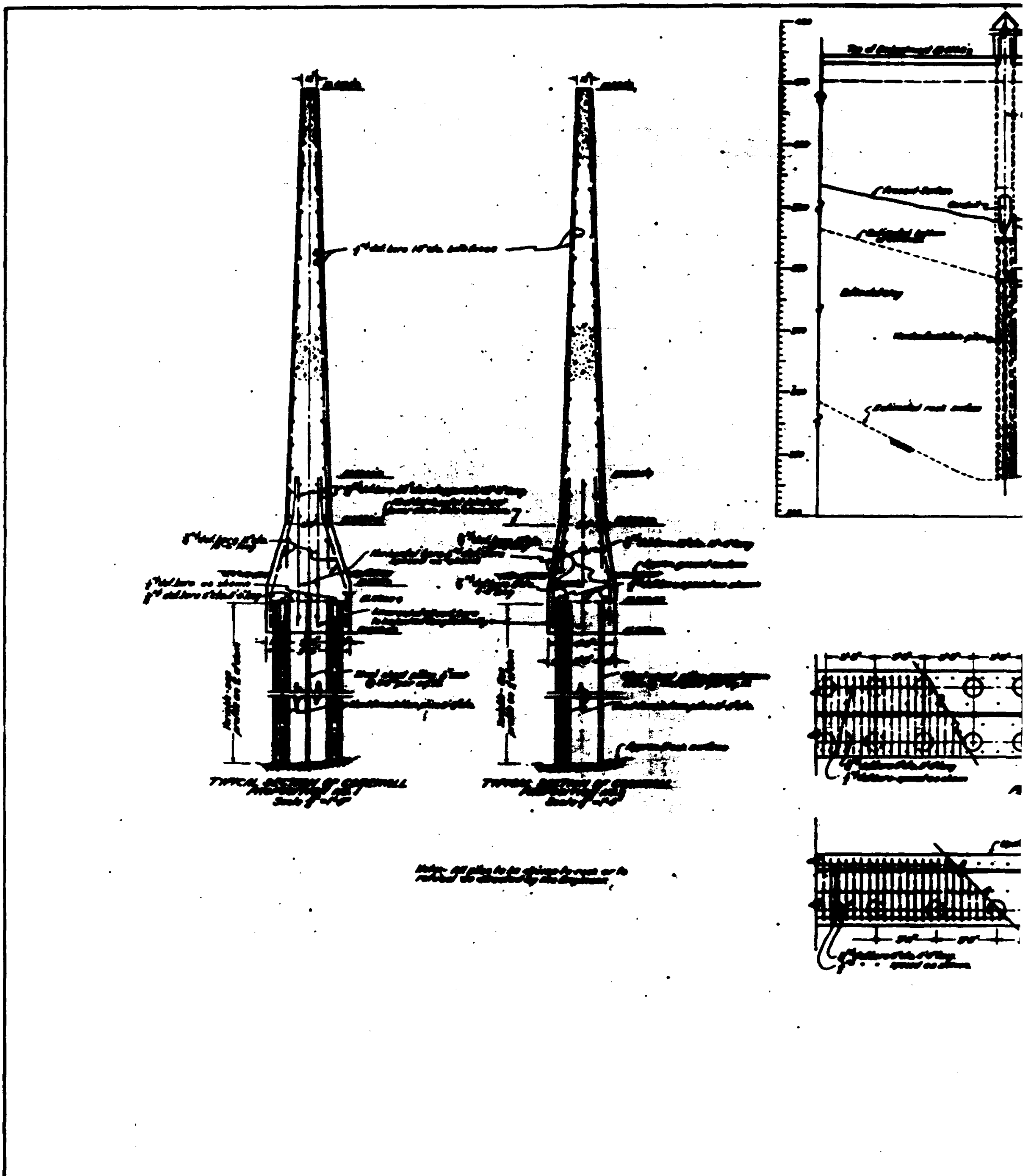
REDUCED TO 47% OF ORIGINAL



NOTE:-
This sheet supercedes sheet No. 10 of the contract drawings for piling and foundation reinforcement only.

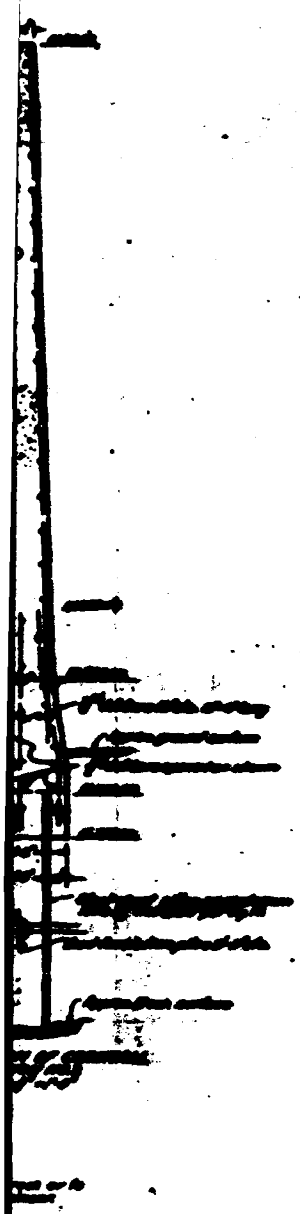
CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
NORTON BROOK DAM AND RESERVOIR
REVISED DETAILS OF
SPILLWAY FOUNDATION

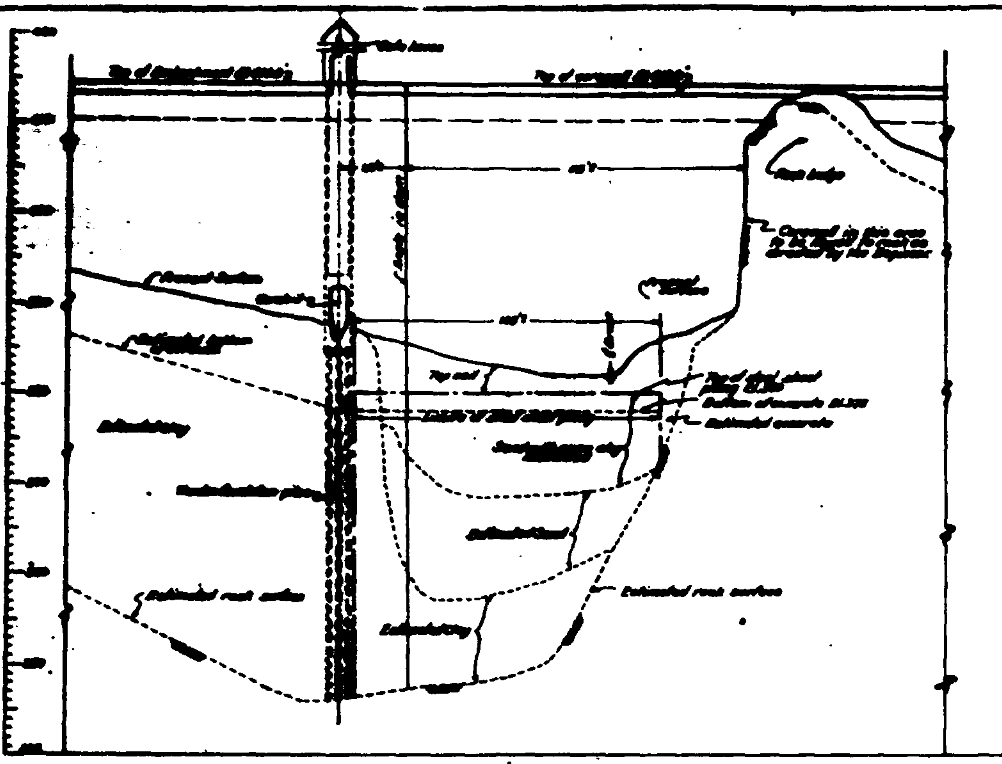
SCALE AS INDICATED



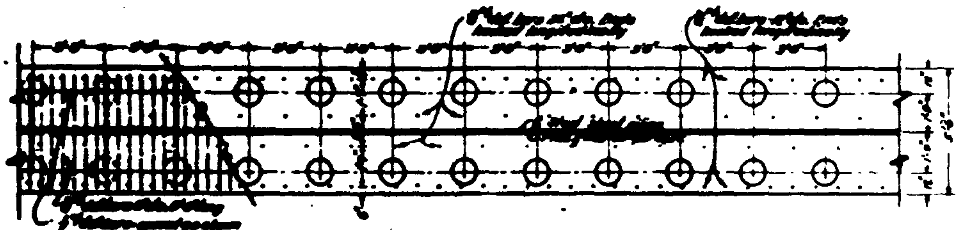
Note:-
This sheet supersedes the profile ... as shown on sheet No. 3 of the contract ... and also supersedes the typical section of corewall as ... on sheet No. 12 of the contract drawings.

2

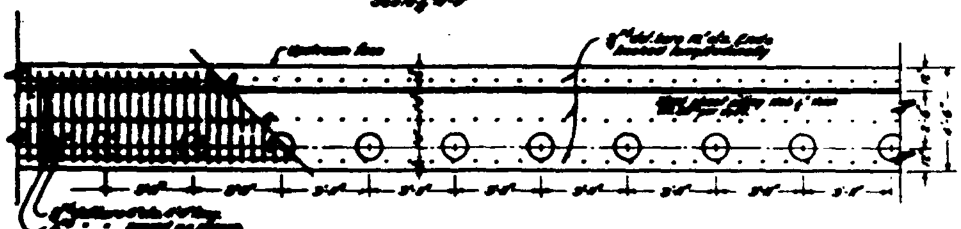




PROFILE ON ℄ OF DAM



PLAN OF COREWALL FOUNDATION
PROPOSITION NO. 1



PLAN OF COREWALL FOUNDATION
PROPOSITION NO. 2

CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
NORTON BROOK DAM AND RESERVOIR
REVISED DETAILS OF FOUNDATION FOR COREWALL BETWEEN SPILLWAY STRUCTURE & ROCK LEDGE

FEBRUARY 1928

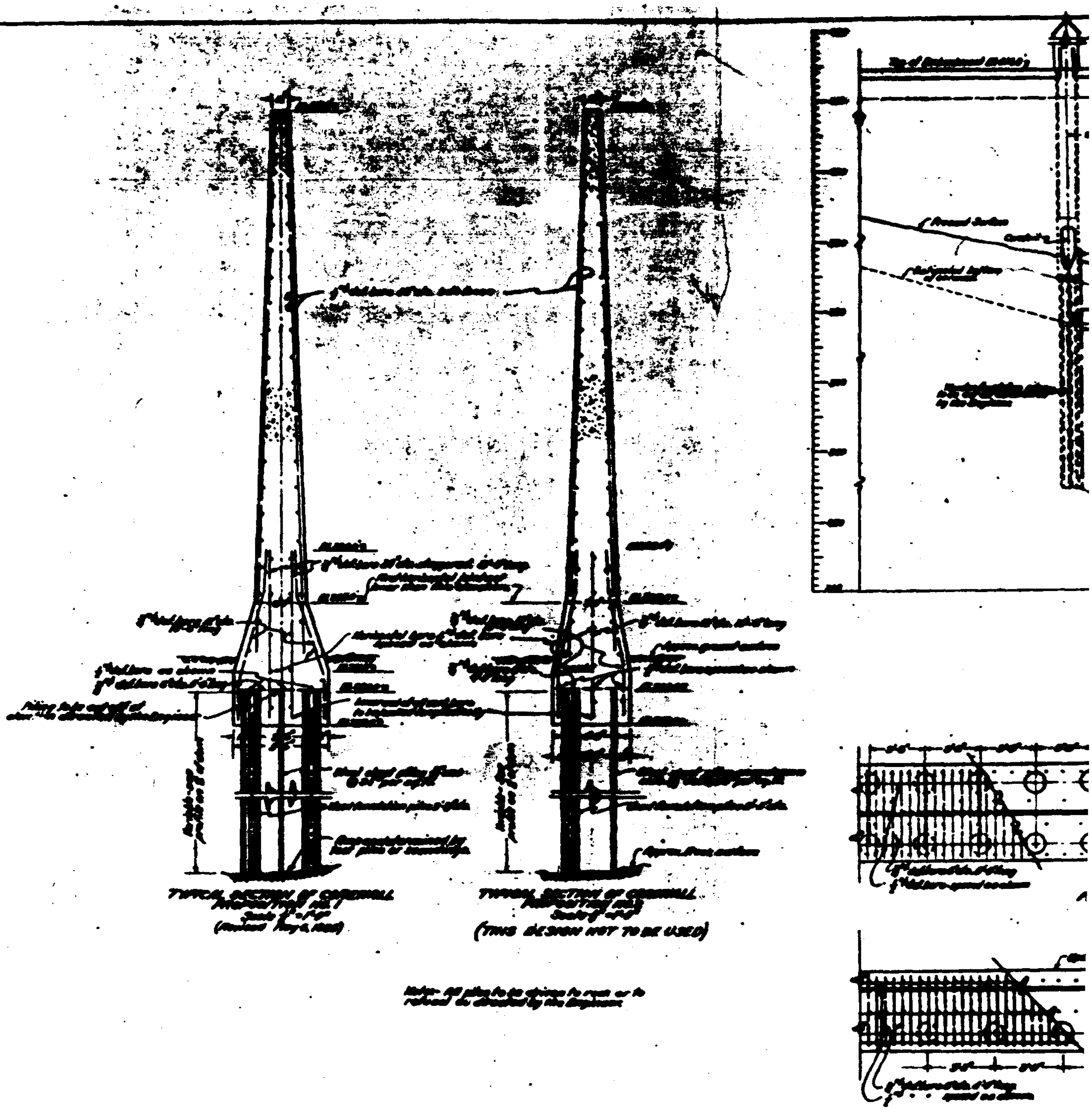
SCALE AS INDICATED

Note:- This sheet supersedes the profile on ℄ of dam between the limits shown as shown on sheet No. 9 of the contract drawings and also supersedes the typical section of corewall as shown on sheet No. 12 of the contract drawings.

B

INC 79-382

B2-10

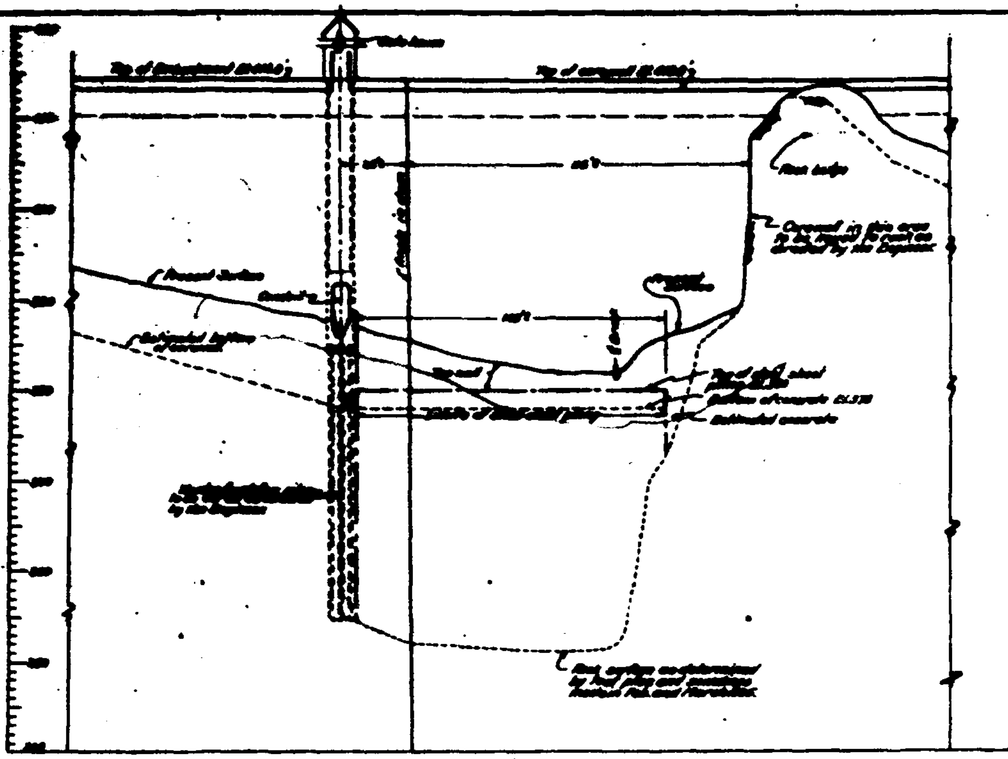


NOTE:— In accordance with letter dated May 2, 1936 from H. R. Rodigan (Engineer Examiner for the State Engineer P.W.A.) the corewall and corewall foundation shall be constructed as detailed by Proposition No. 1 on this sheet.

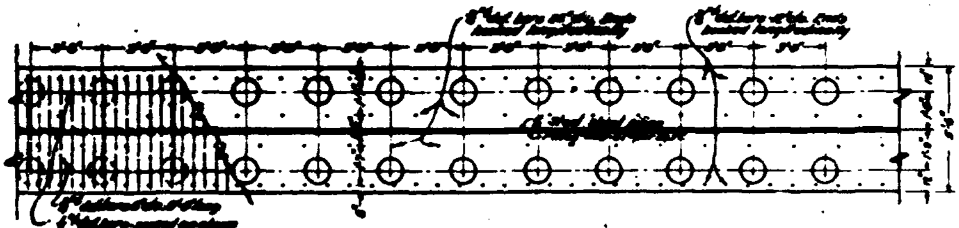
P.W.A. PROJECT No. 597

REDUCED TO 50 % OF ORIGINAL

GORDON E. AINSWORTH & ASSOCIATES INC 79-382



PROFILE ON ℄ OF DAM
(Revised May 6, 1935)



PLAN OF COREWALL FOUNDATION
PROPOSITION NO. 1
(Revised May 6, 1935)

PLAN OF COREWALL FOUNDATION
PROPOSITION NO. 2
(THIS DESIGN NOT TO BE USED)

CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
NORTON BROOK DAM AND RESERVOIR
REVISED DETAILS OF FOUNDATION FOR COREWALL BETWEEN SPILLWAY STRUCTURE & ROCK LEDGE

FEBRUARY 1935
SCALE AS INDICATED

Note— This sheet supersedes the profile and ... shown on sheet No. ... of the contract drawings and also supersedes the typical section of corewall as shown on sheet No. 12 of the contract drawings.

Revised May 6, 1935.

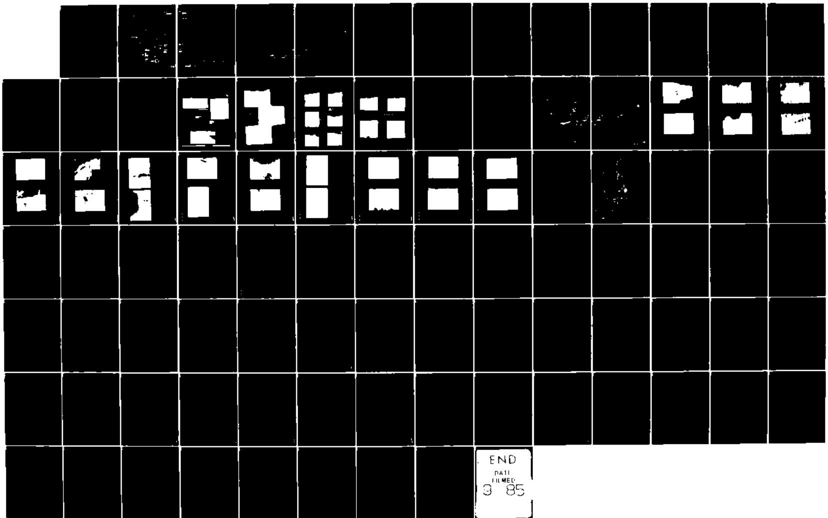
BARKER AND WHEELER, ENGRS.
36 STATE ST., ALBANY, N.Y.
11 PARK PLACE, N.Y. CITY.

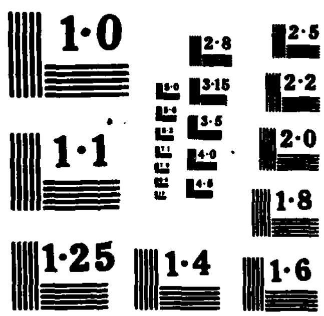
B

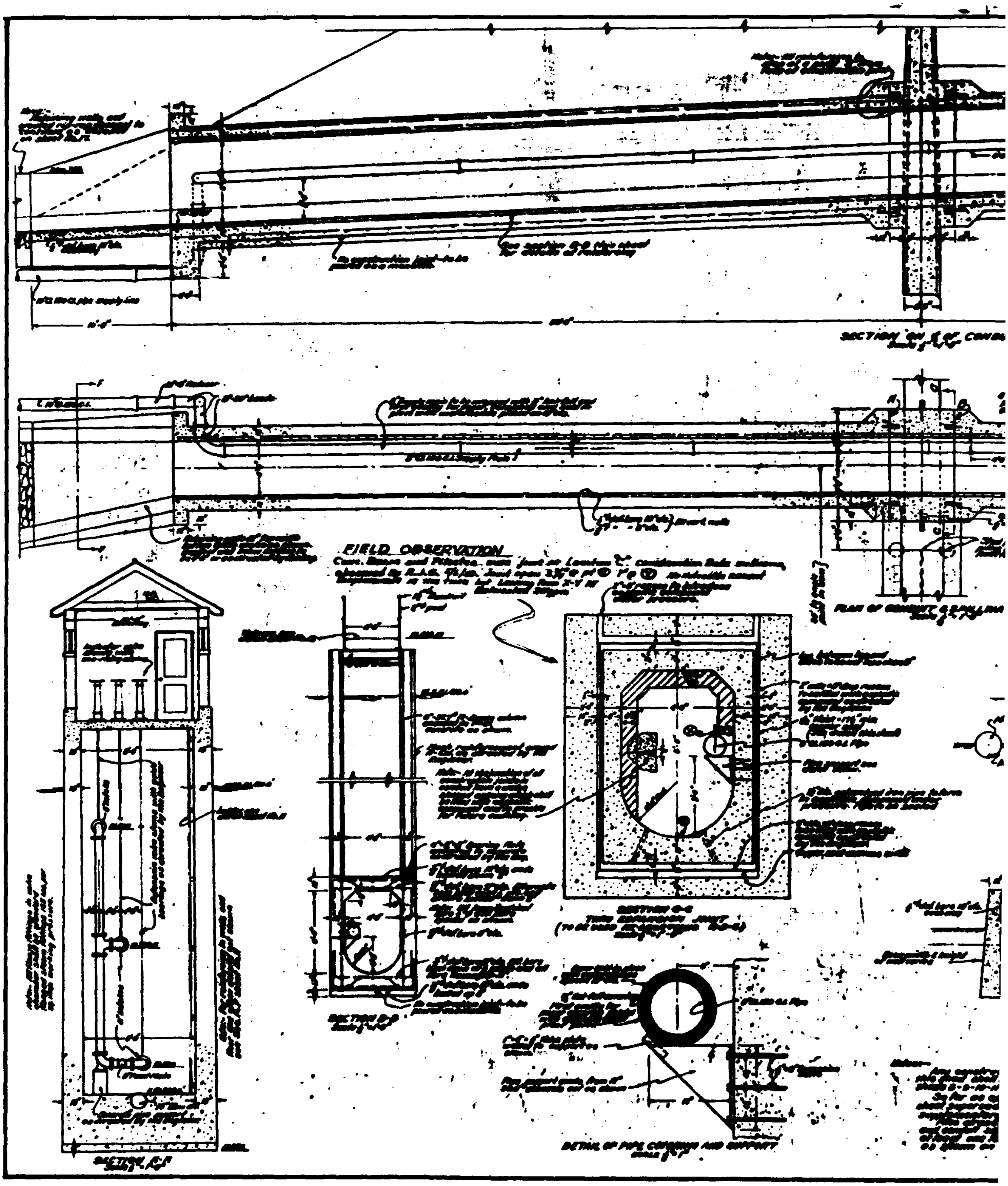
AD-A157 626 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS NORTON BROOK DAM (VT..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV JAN 80

1/2

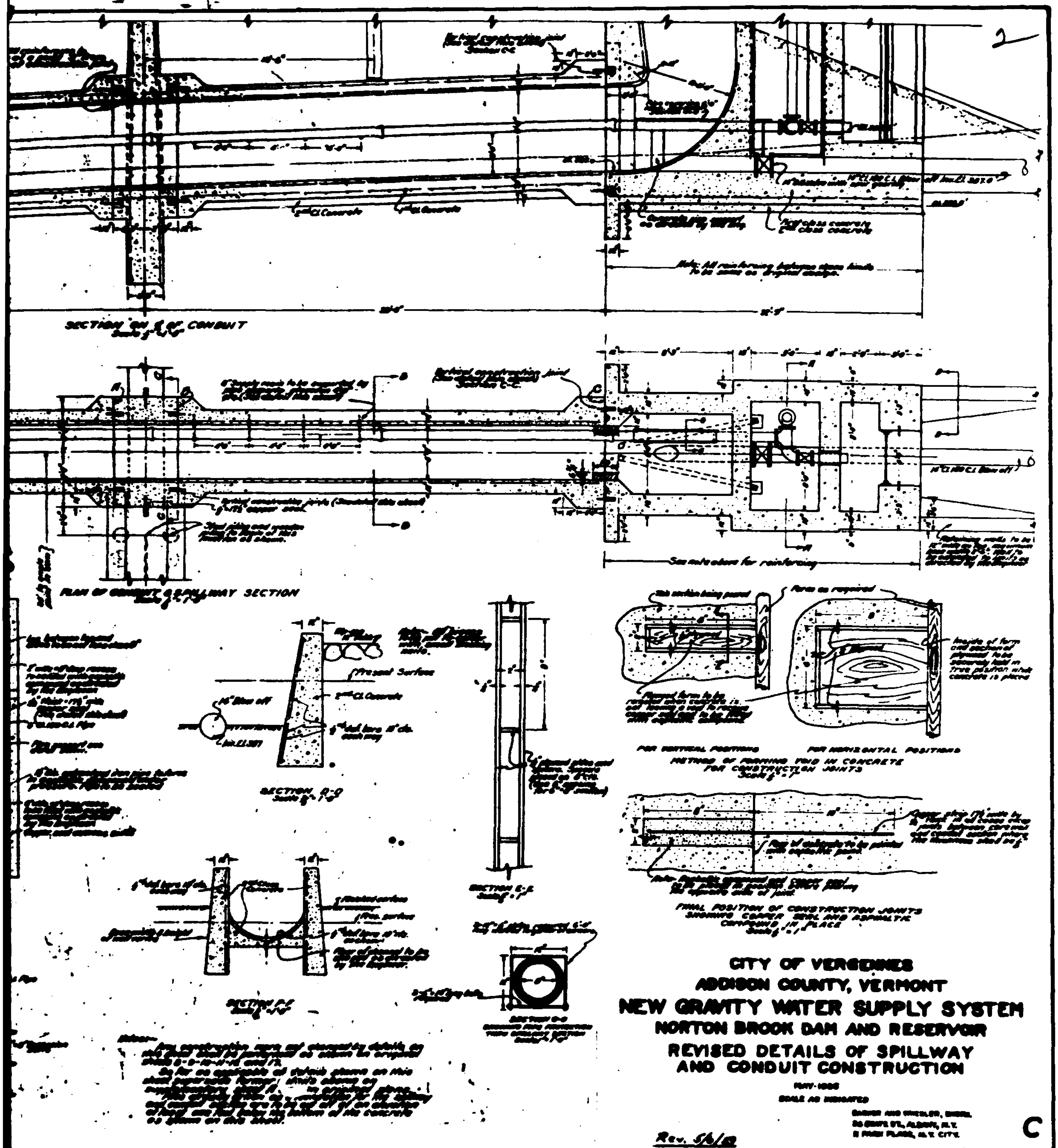
UNCLASSIFIED F/G 13/13 NL




1·0
2·8
2·5
3·15
2·2
3·5
2·0
1·1
4·0
1·8
1·25
1·4
1·6

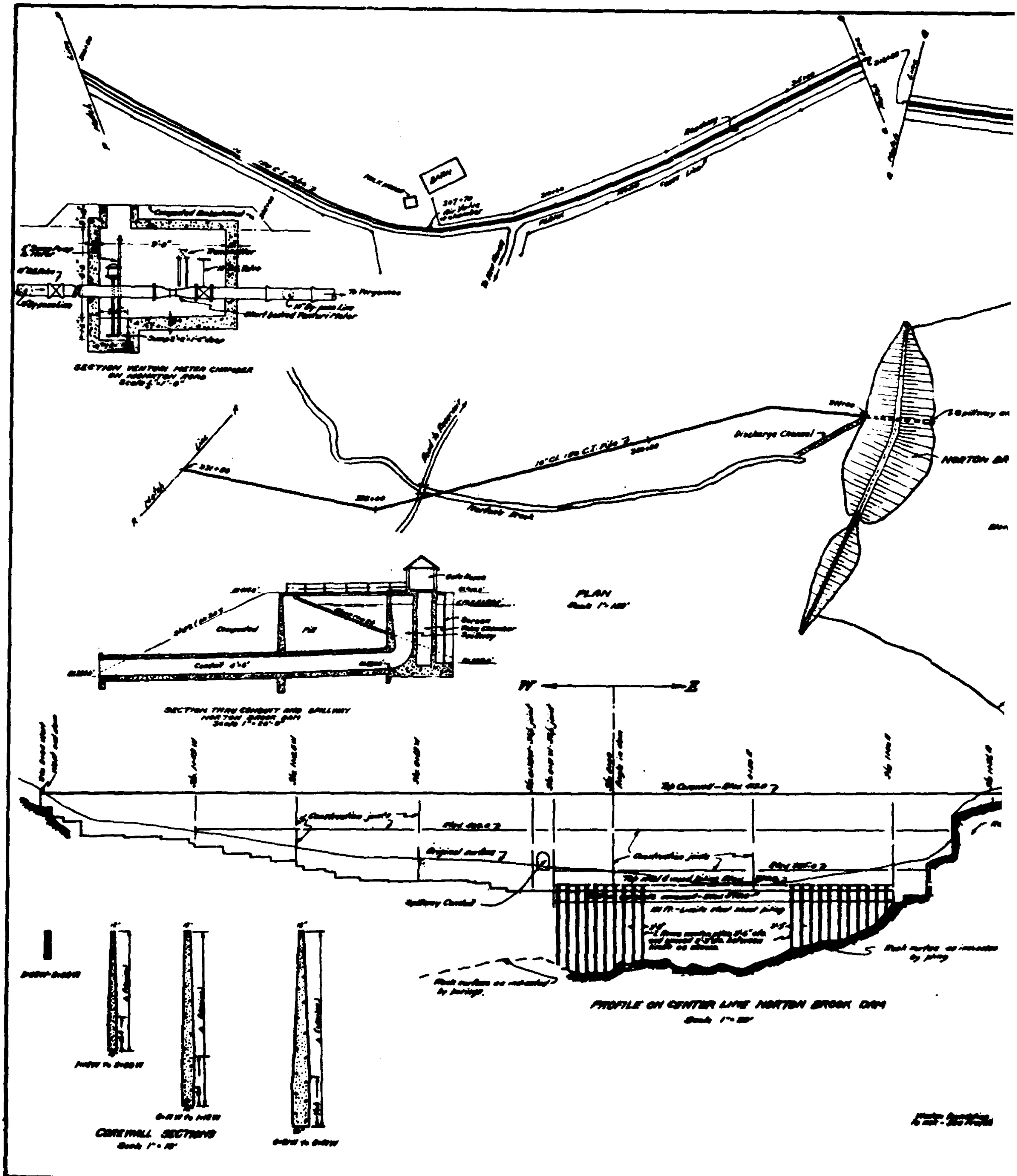


REDUCED TO 47% OF ORIGINAL



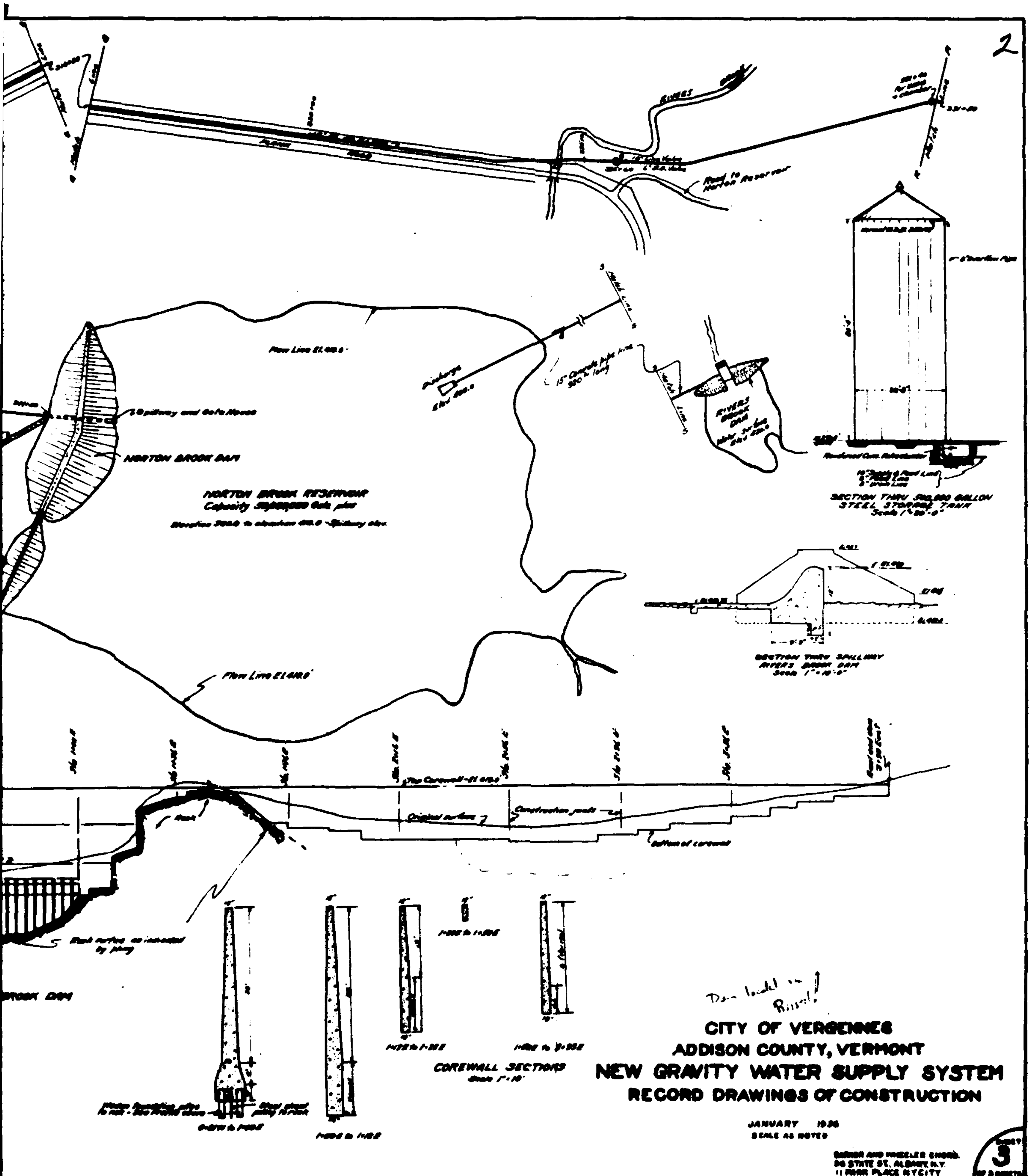
ES INC 79-382

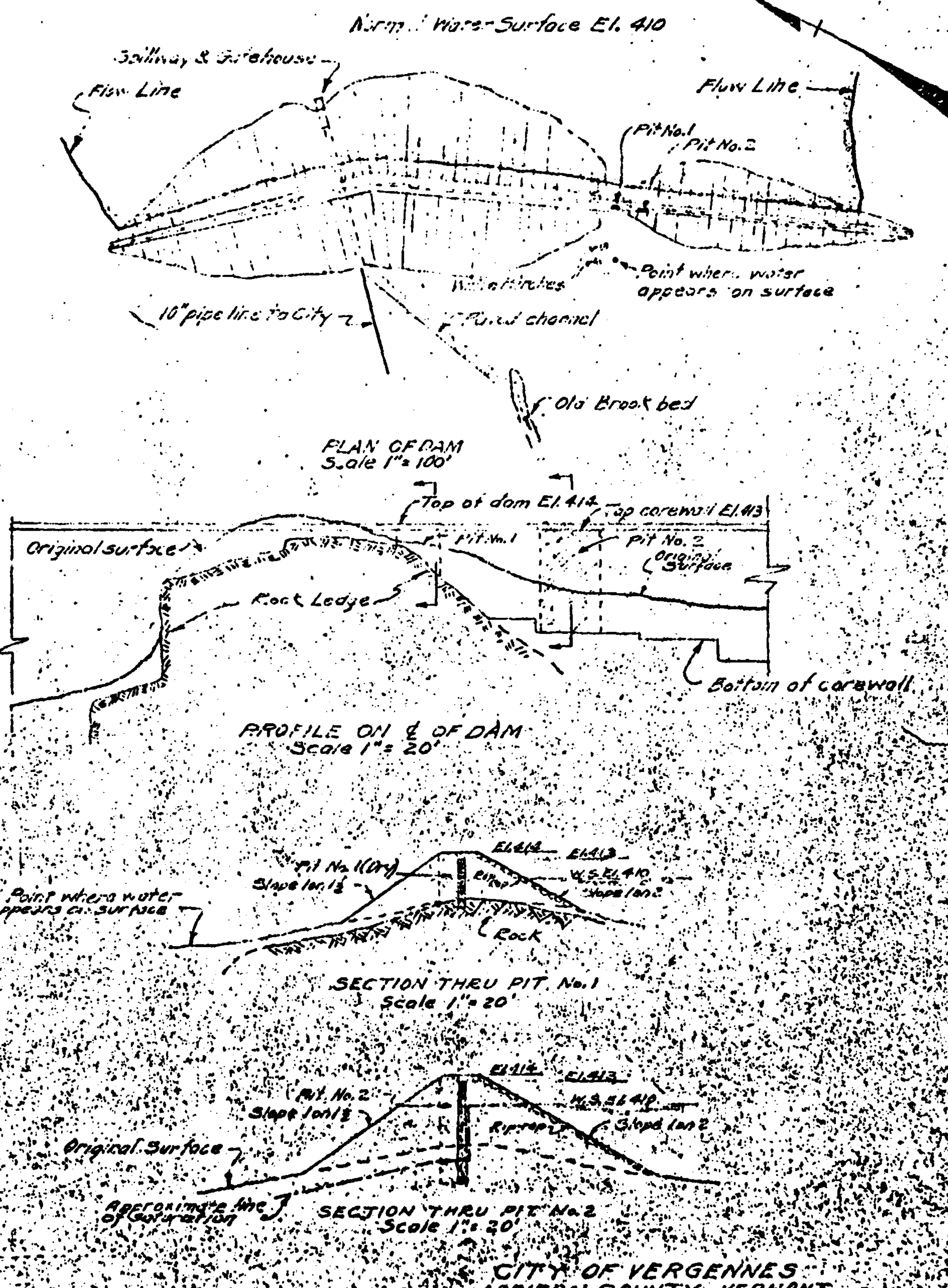
B2-12



REDUCED TO 47 % OF ORIGINAL

GORDON E. AINSWORTH & ASSOCIATES INC 79-382


2
Road to Norton Reservoir
Flow Line El. 416.0
Discharge
15" Concrete pipe line 380' long
Spillway and Gate House
NORTON BROOK DAM
NORTON BROOK RESERVOIR
Rivers Brook Dam
Section thru 500,000 gallon steel storage tank
Section thru spillway Rivers Brook Dam
Flow Line El. 416.0
Top Corewall
Original surface
Construction joints
Bottom of corewall
Rock
Rock surface as indicated by piling
Brook Dam
Corewall Sections
Do-- lo-k-l -- Rissel!
CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
RECORD DRAWINGS OF CONSTRUCTION
JANUARY 1936
SCALE AS NOTED
3



CITY OF VERGENNES
ADDISON COUNTY, VERMONT
NEW GRAVITY WATER SUPPLY SYSTEM
PLAN AND SECTIONS OF NORTON BROOK DAM EAST OF ROCK LEDGE WHERE WATER IS APPEARING AT ORIGINAL SURFACE BELOW DAM

JULY 1936
SCALE AS NOTED

BARKER & WHEELER ENGRS.
36 STATE ST. ALBANY, N.Y.
11 PARK PL. N.Y. CITY

B2-14

APPENDIX B

Section B3

COPIES OF PAST INSPECTION REPORTS AND DATA

TABLE OF CONTENTS

	Page
Inspection Report on Norton Brook Dam - November 1951	B3-1
Letter by Barker & Wheeler Engineers to the Vermont Commission of Water Resources - September 24, 1957	B3-2
Inspection Report by Barker & Wheeler Engineers to the City of Vergennes - September 20, 1957, and accompanying photographs	B3-3 to B3-13
Note on Dam Inspection by John E. Cerutti - October 10, 1957	B3-14

A/2
1/11

INSPECTION REPORT
ON
Norton Brook Dam

1. Date of inspection Nov. 1951 2. Water conditions normal

GENERAL DATA:

3. Location of dam Norton Br.; Town of Bristol
4. Owner and operator City of Vergennes
5. Characteristic features of dam embankment dam with drop inlet type spillway
6. Other related data (see PSC case # 1824) Serves as a water supply

OBSERVATIONS:

7. Condition of structure embankment - shows settlement particularily at eastend; good grass cover; soggy toe; shows seepage at foundation line. Concrete outlet structure - in good condition; some scour in open section of outlet channel
8. Condition of equipment good
9. Operation good
10. Maintenance good

REMARKS: Partial failure occurred at time of 1938 flood. Structure is located in an isolated location.

Inspected by RHH

NOV-2 1979

BARKER & WHEELER

Engineers

36 STATE ST., ALBANY 7, N.Y.

September 24, 1957.

Mr. R.W. Thieme,
Commissioner of Water Resources,
Montpelier, Vt.

Dear Mr. Thieme -

I am sending you copy of a letter written to Vergennes. I am going to St. Albans and will stop at Vergennes on the way to inspect the work which has been done in clearing the down-stream face of the dam preliminary to making an investigation of the saturation.

I called Carroll Blair, Commissioner of Public Works, yesterday, and he told me that he already had a good start on the clearing - perhaps two-thirds done, so I will stop and see it, and will let you know how soon it will be in condition to have Mr. Cerutti meet me there for an inspection and discussion as to the best method of continuing the investigation.

I am sending you enclosed ~~under separate cover~~ a copy of the Rutland report. I will probably be in Brandon and Rutland Wednesday or Thursday. I expect to meet with the Selectmen in Brandon, and go over the sewage disposal situation with Mr. Frank L. Rice, Commissioner of Public Works at Rutland.

Sincerely,

Robert C. Wheeler

Robert C. Wheeler

RCW:mwc

RECEIVED
SEP 25 1957
Water Conservation Board

WATER SUPPLY — SEWERAGE — SEWAGE DISPOSAL — WASTES REMOVAL — POWER SYSTEMS
DESIGN, CONSTRUCTION AND OPERATION OF MUNICIPAL, PUBLIC UTILITY AND INDUSTRIAL PLANTS

BARKER & WHEELER

Engineers

36 STATE ST., ALBANY 7, N.Y.

September 20, 1957

C O P Y

Hon. Alan W. Wright, Mayor and
Board of Aldermen,
Vergennes, Vt.

Gentlemen:

In accordance with your request as conveyed to me by City Clerk, Eldon Griffith, on September 10th in the company of Public Works Commissioner, Carroll W. Blair, I made an inspection of the Norton Brook Dam and Reservoir, having special reference to the settlement of the dam and the seepage at the toe of the earth embankment.

I found the situation serious, but apparently not of a nature to demand emergency measures, but certainly one which would not permit of delay.

I asked Commissioner Blair to clear all brush and undergrowth from the dam so that complete inspection and investigation could be made, clearing the entire site above the fence at the toe of the dam and outside the fence to the corrugated culvert.

I suggested that the trees be left on the dam for the time being, not because they should not be removed, but because they would not interfere with the investigation, and because they should be removed in such a manner as to cause as little damage to the earth embankment as possible.

In the construction of the dam, the earth embankment was carefully placed in layers and compacted and rolled, and there should not be any growth allowed on it, except a sod cover to prevent washing by rain.

Undergrowth on the slopes of the dam prevents proper inspection of the condition of the earthwork. This is particularly troublesome now when it is necessary but impossible to ascertain the exact condition of the embankment. Larger trees tend to loosen the embankment through the extension of their roots and make it more porous, and less able to hold water and resist the pressures against the dam.

The larger the trees grow, the more of a hazard they are, particularly at times of high wind when the roots must react and move in conformity with the action and movement of the tops, and thus do further damage to the embankment. An extreme example would be a hurricane in which the trees were laid flat with their roots in the air and part of the embankment clinging to them.

It was understood from the beginning that this dam was difficult of construction and would require careful maintenance, and these matters have received a great deal of consideration over since. However, it is some 22-years since the dam was built and many of the people who were intimately connected with its construction are no longer available for consultation.

I am sending under separate cover a set of five plans which depict the salient points of the structure, and include in this memorandum a brief historical resume outlining the construction and some of the investigations carried out as the reservoir went into service.

The plans are as follows. Sheets 8 and 9 of the contract drawings simply give the layout of the dam and reservoir as a unit, with 9 indicating the sections through the dam.

Sheet C gives certain revised details of the spillway and the conduit through the embankment and corewall which carries the runoff, and serves as a housing for the pipe lines through the dam.

Sheet 3 is one of the sheets of record drawings which shows in part the rock encountered and the piling driven. It will be noted that the corewall detail under the high part of the dam, was changed so as to locate the steel sheet piling under the center of the corewall, while the corewall was further supported by wooden foundation piles driven to rock and on both sides of the steel piling.

There is also a sheet dated July 1936 which indicates the work done in investigating the source of water which appeared on the surface just southeast of the white birch trees shortly after the dam was completed. Two test pits were dug to the bottom of the

corewall at points opposite where the water was observed on the slope downstream from the dam, and perhaps 10-ft. south and east of the white birches indicated on the plan.

History of the Dam

The dam was built in 1935 under PWA grant. At the close of this work, before the final payments were ultimately approved, a thorough check was made, beginning in the Spring of 1936, to ascertain whether or not there was any water coming through the dam. Certain quotations will be given from correspondence, ranging through the years, which indicate that this was under consideration prior to the time when the southeasterly end of the dam was washed out, and again after that break had been repaired.

The dam was built on a foundation that was not altogether satisfactory. This will be evidenced by the presence of the steel sheet piling cutoff diaphragm, and the fact that the corewall was supported on wooden piles extending to rock.

During construction the foundation was observed to yield slightly under the load, so wooden foundation piles were installed on both sides of the center line of the concrete cutoff walls, the base of which was spread, where they were used, from a width of about 36" to 66".

There were also a few places where slight seepage of water was observed before the dam was built, but this was not serious and did not at any time during our observation, appear serious, since the water was not carrying any sediment with it which might tend to weaken the foundation or indicate any erosion of the earthwork after the dam was built.

\- - - - - - - - -

From letter July 1, 1936 written by Mr. Wheeler to Mr. Harrington, Chairman of the Vergennes Water Committee:

> "As a result of this inspection, I feel that the conditions do not give any grounds for uneasiness. The seepage at the west end of the dam back of the cement shed is negligible. The seepage which had occurred below the west end of the ledge at the middle of the dam has been traced to its source in fissures near the bottom of the ledge. It is not carrying any material and gives no grounds for concern, except that it should be watched.
>
> "So long as you do not need the water and the water does not carry any silt, there is no need to take any steps to correct the situation. In addition,

it is quite likely that the fissures will silt up in the course of time and the flow diminish or altogether cease.

"The one remaining location which might deserve further thought is the bubbling of water out of the ground which occurs near the white birch trees at the easterly edge of the ledge. It must be remembered that this spring was flowing in the natural ground some fifteen or twenty feet away from the toe of the slope of the dam embankment."

Letter July 2, 1936 from our Mr. Hall to W. E. Boothby who had been Resident Engineer:

"The Inspection Division at Concord are inclined to think that the water east of the white birches and the ledge, may be coming under the corewall. We have examined the pit on the dry side of the corewall and the water came in very slowly, to a depth of about 3-1/2 ft, and did not rise any further, although there was a head of about 7-1/2 ft on the reservoir side."

July 9, 1936, Memo to Mr. Wheeler from Mr. Hall:

In order to give opportunity to measure the flow from this source near the white birches, a weir plate 1/8" thick and 12" long was set in concrete. The concrete sidewalls are 3" above the top of the weir plate. It was observed that at times the flow over this plate was as much as 1/2", or about 20,000 gpd. However, it varied with the rainfall, rather than with the depth of water in the reservoir.

July 10, 1936, a letter from Mr. Wheeler to PWA inspector conveyed the information that no sediment was carried by any water coming from this source, and it did not have any practical significance. The communication said, however, that the condition should be watched.

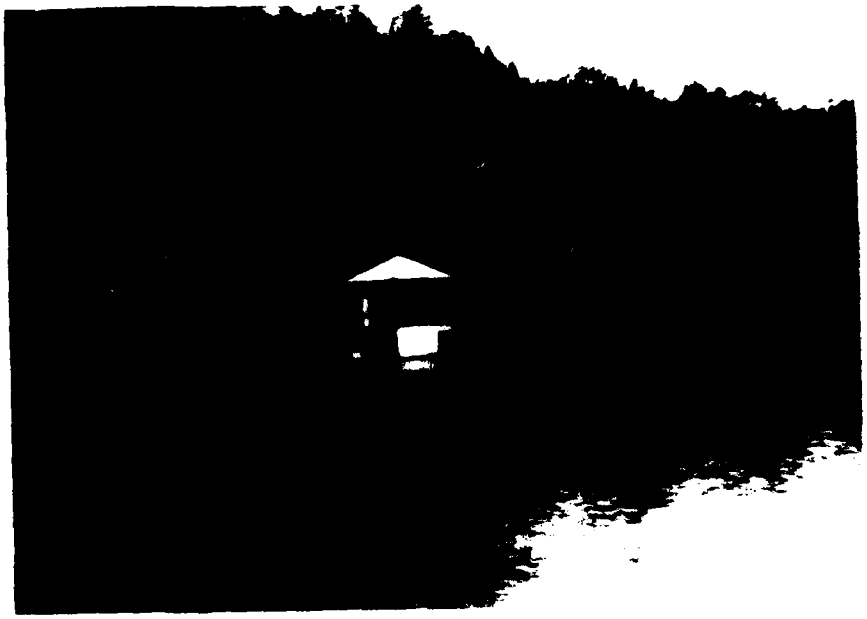
July 28, 1936, a letter from Mr. Harrington to this office indicated that the flow over the new weir which was installed on July 8th, and by that time had been in operation some three weeks, remained about 1/2", and after a rain increased to 3/4".

C-4A Trees and brush on downstream slope of dam with left abutment
(rock outcrop) in the background - 10/24/79

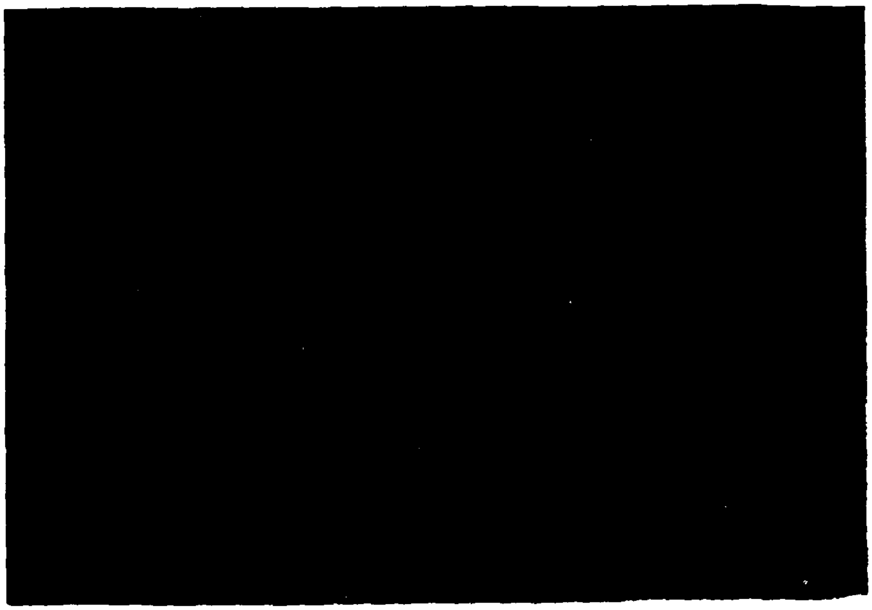
C-4B Trees on lower part of downstream slope of dam embankment
10/24/79

C-3A Dam crest looking from sta 0 + 00 (angle point) toward right abutment. Note service bridge railing through trees - 10/24/79

C-3B Upper part of downstream slope of dam embankment looking from sta 0 + 50R toward left abutment - 10/24/79

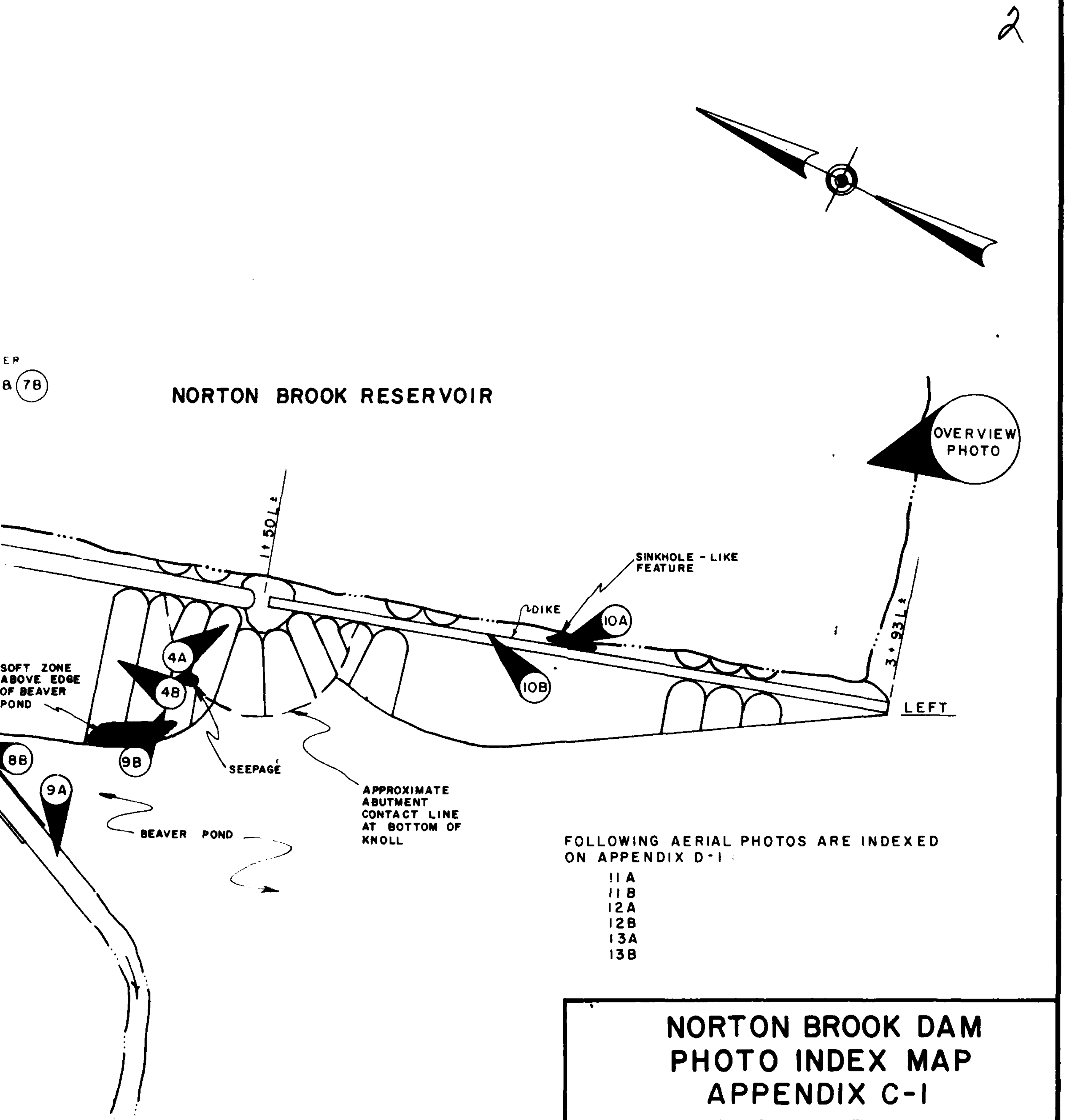


C-2A Control tower, crest and upstream slope of embankment looking from upstream of right abutment - 10/24/79



C-2B Aerial view of downstream slope of embankment - 11/30/79

2



FOLLOWING AERIAL PHOTOS ARE INDEXED ON APPENDIX D-1

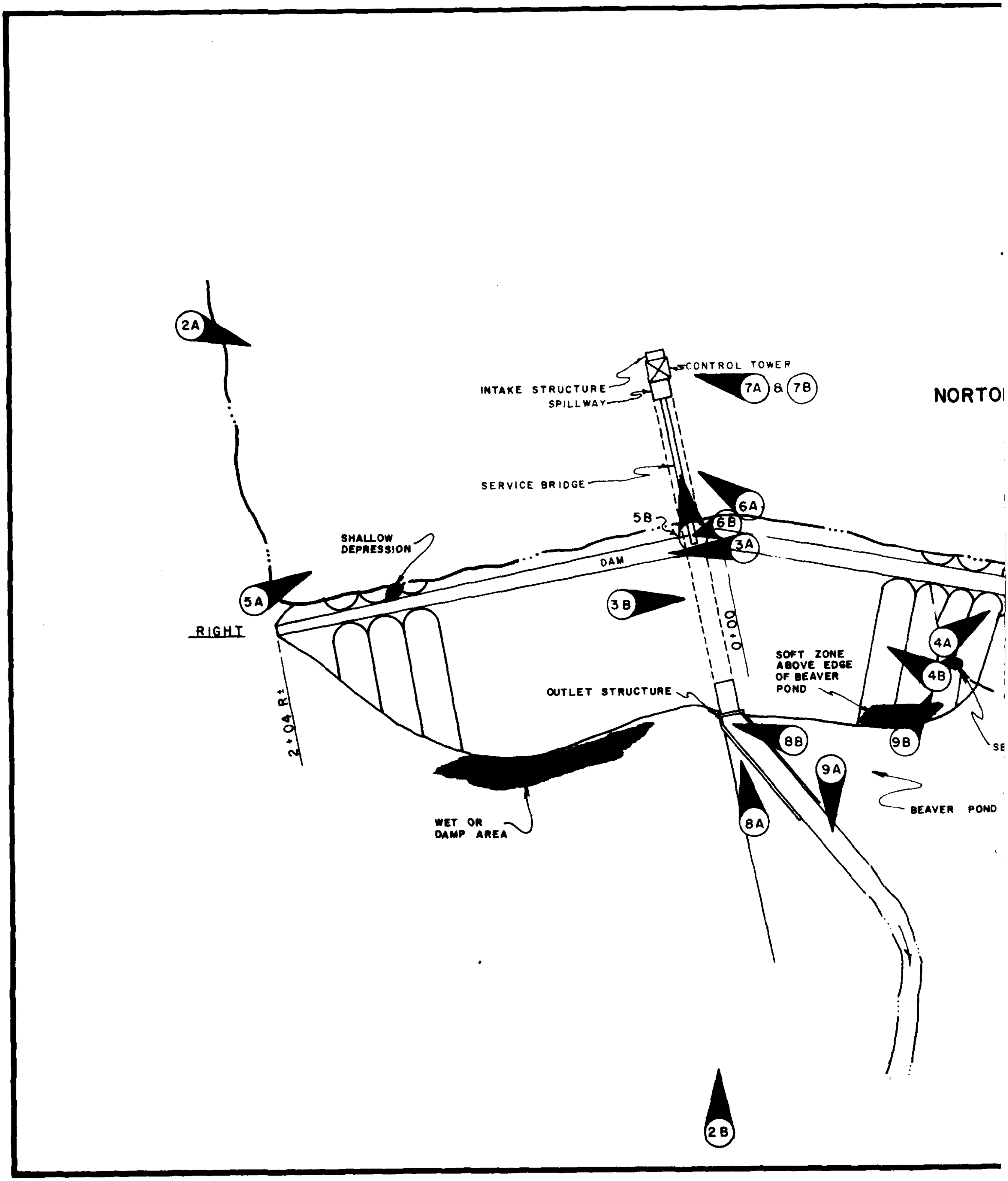
11 A
11 B
12 A
12 B
13 A
13 B

SCALE : NONE

NORTON BROOK DAM
PHOTO INDEX MAP
APPENDIX C-1

GORDON E. AINSWORTH & ASSOCIATES, INC.
Engineers, Surveyors and Planners
30 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

DWG. NO. 79-383



NORTO
CONTROL TOWER
INTAKE STRUCTURE
SPILLWAY
7A & 7B
SERVICE BRIDGE
6A
5B
6B
3A
SHALLOW DEPRESSION
DAM
2A
5A
3B
RIGHT
2+04 R±
0+00
SOFT ZONE ABOVE EDGE OF BEAVER POND
OUTLET STRUCTURE
4A
4B
8B
9B
9A
BEAVER POND
WET OR DAMP AREA
8A
2B

APPENDIX C

PHOTOGRAPHS

A(2)

October 10, 1957

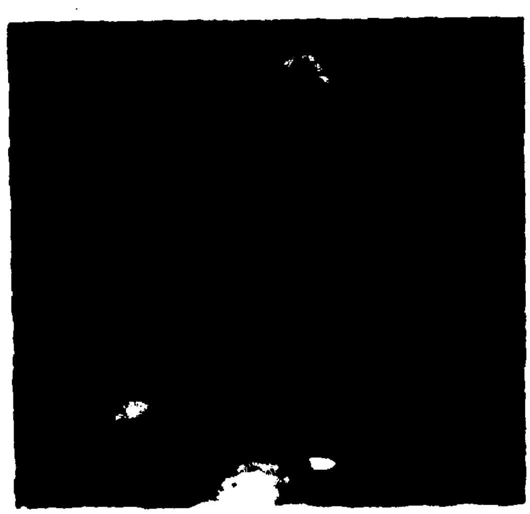
The undersigned and Donald Webster along with Robert Wheeler, Mr. Frazier and Carroll Blair visited the Vergennes water supply dam and reservoir on October 10, 1957. The condition of the dam is the same as reported by Robert Wheeler in reports we have received from him except that all the brush has been cleared downstream of the dam. The downstream side of the dam has evidently dried out some since the brush has been cut off. Mr. Wheeler asked Mr. Blair to have some holes bored on the downstream side of the core wall about 50' apart to check the ground water level on the downstream side of the core wall.

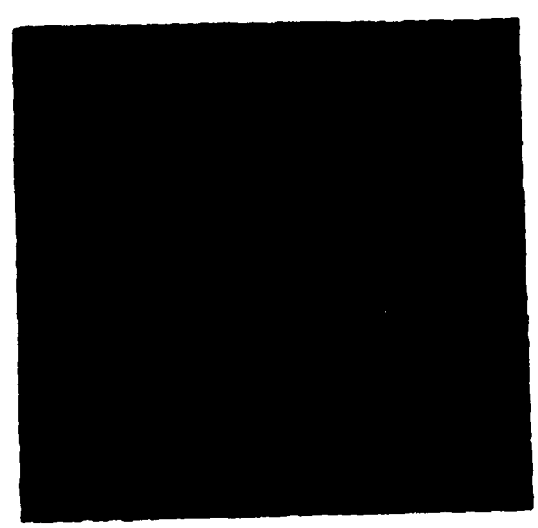
S/ John E. Cerutti
Hydraulic Engineer

Norton Brook Dam, Vergennes, Vermont - September 1957

Dam, looking northwest

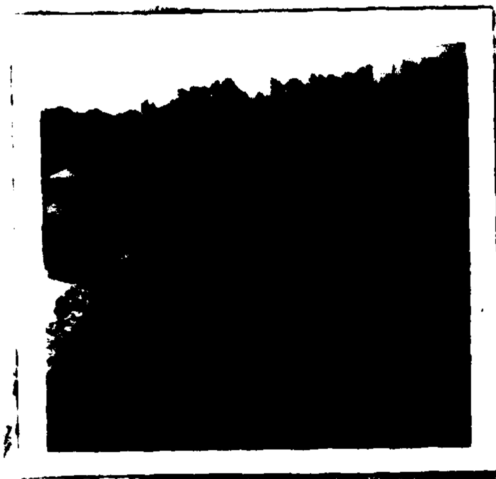
Culvert and seepage below dam

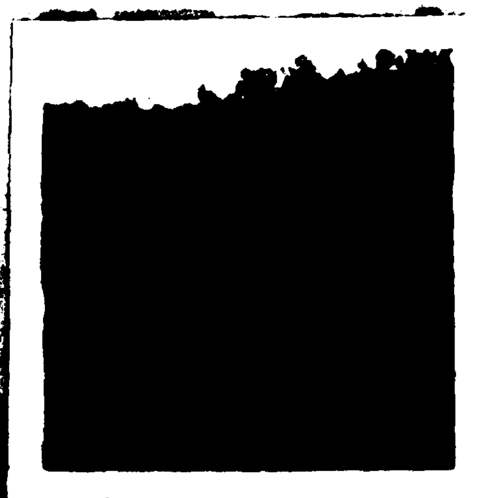


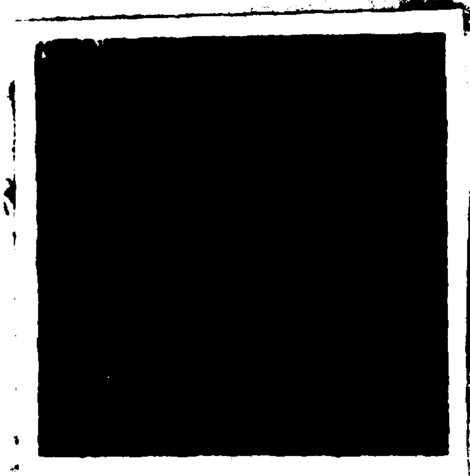


Norton Brook Dam, Vergennes, Vermont - September 1957

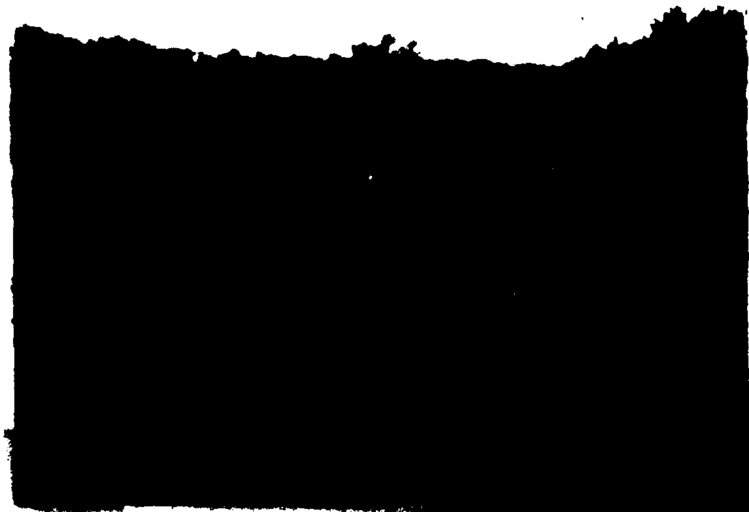
Dam, looking southeast





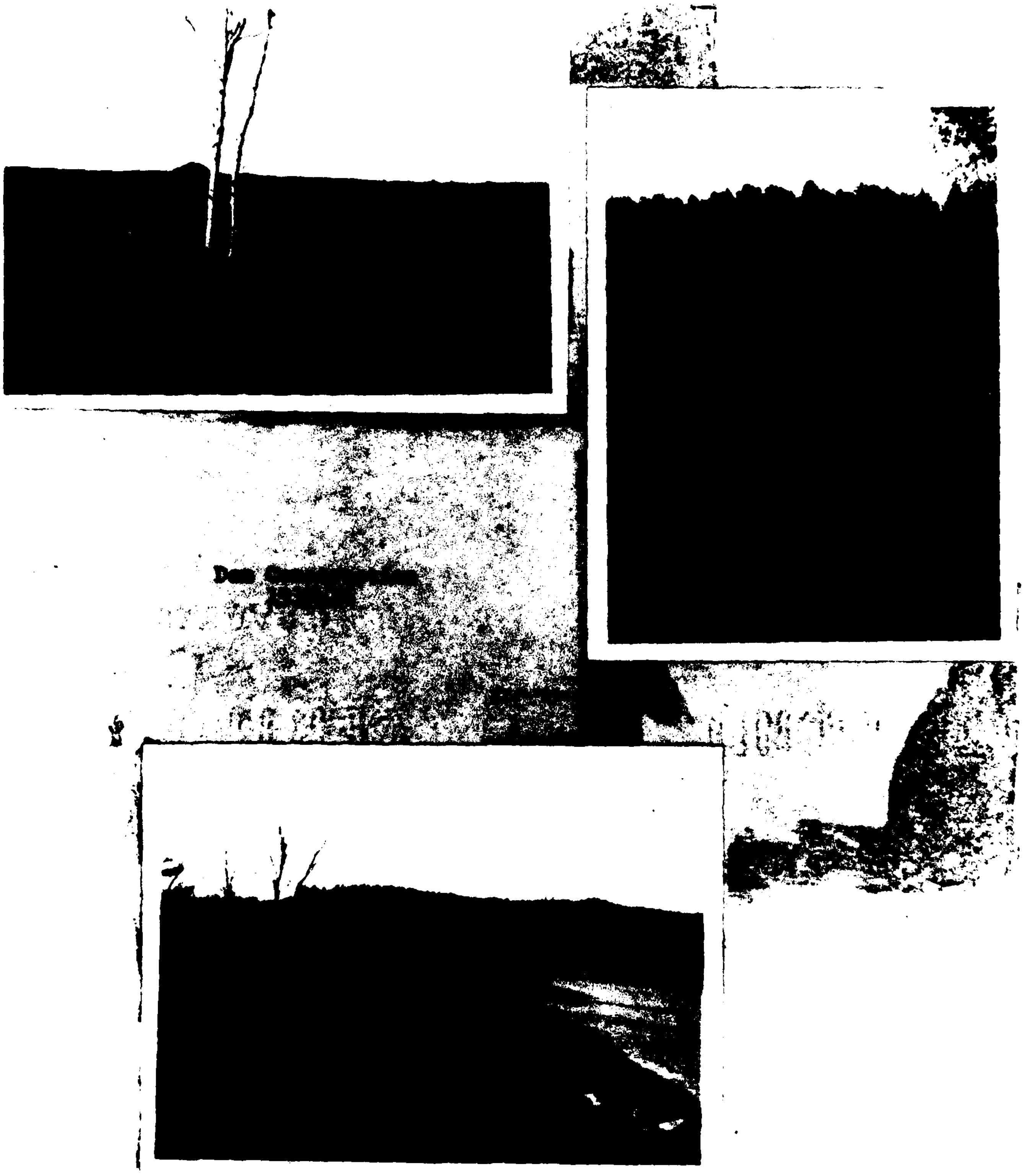


Norton Brook Dam, Vergennes, Vermont



Showing washout and east end of Dam June 1942

Norton Brook Dam, Vergennes, Vermont



In addition, the toe of the slope has become so saturated that it has sluffed down for a considerable distance. However, the dam faces - both on the upstream and the downstream side - are so grown up to undergrowth, shrubs, small trees, and even sizable pines, that it is impossible to form any judgment as to the volume of water and where it comes from until these obstructions have been cleared away.

I have recommended to Mr. Blair that he proceed at once to clear out the smaller growth and mow the grass and weeds, and I will come up and inspect it again.

A thorough investigation should be made as to the extent to which the embankment material is saturated, so that steps can be taken to have it dried out. At present its resistance to any force acting upon it has been appreciably weakened. We are investigating similar conditions at present in connection with a dam built of concrete. The stability of the dam should be established before any definite pressure is brought to bear on it. Test drillings should be made, test pits dug, or both; and the zone and extent of saturation established.

I would appreciate it if the clearing were taken care of as soon as possible, since I expect to be in Vermont next week, and would like to see Mr. Blair, and possibly meet with the Board at that time. We could then determine the program to be followed. I will let you know in advance, but it will probably be the middle of next week.

It was expected that this letter would be accompanied by a considerable number of pictures which illustrate thevarious points raised. It has not been possible to secure these prints, and I will try to bring them with me when I come.

Sincerely,

Robert C. Wheeler

RCW:mwc

Cc Mr. Blair

July 8, 1942 - Letter from Mr. Wheeler to Hon. William E. Larrow, Mayor:

"There is one point to which I wish to invite the attention of yourself and the other officials, and that is that an earth structure is of a perishable nature and has to be maintained. With proper maintenance it should last indefinitely; however, your dam has been allowed to grow up to weeds, thistles, berry bushes and even small trees. The effect of tree roots in the dam is to tend to loosen the earth.

"The dam should be kept moved and inspected periodically for holes from settlement or erosion, or possible cracks that might form when the dam is drawn down, or even holes made by animals, and these should be repaired promptly.

"When the dam is filled again, it should be carefully inspected as the water level is raised.

"Weirs should be placed so that they will indicate the flow of the springs immediately below the dam, as they were before, and accurate readings should be made periodically so as to detect any increase in flow. They should also be so arranged that there would be a pool lined with concrete just behind the weir in which it would be possible to detect immediately when any soil or other material is being carried by the flow of the water. These weirs should be placed as close as possible to the point of the outcropping of the water."

November 15, 1944 - Letter from Mr. Wheeler to George Stone, City Clerk:

"The dam should be properly maintained. It should not be allowed to be overgrown with brush and observations should be made at regular intervals of the amount of water in the seepage below the dam."

From the foregoing, it will be seen that the amount of water appearing at the toe of the slope on the downstream side of the dam has been a matter of concern ever since the dam was constructed.

At the time of my recent inspection (September 10, 1957), I found the embankment of the toe of the slope on the downstream side of the dam more saturated than I ever recall seeing it, and the flow at that point seems to be greater. A corrugated iron culvert has been installed in the old stream bed below the dam.

October 20, 1936, letter from Mr. Harrington to Mr. Wheeler stated as follows:

"The water flow over the weir that Mr. Hall placed on the spring at the outside of the dam, has not been flowing as fast as it did. I would say that the flow is not over 1/3", perhaps 3/8". (This is equivalent to about 13,000 gpd.)

On June 21st, 1942 part of the southeasterly end of the dam to the south of the rock ledge washed out, exposing and undercutting the corewall.

The exact cause of this washout has never been determined, but it seemed to be the result of a combination of factors which included the neglect of the condition of the dam slopes, and a growth of vegetation on them, which prevented regular inspection and maintenance. The reported raising of the spillway increased the pressures on the earthwork and subjected areas to flooding, which had not been previously flooded.

If there had been holes made by rodents, these could well be the source of the entrance of water into the earthwork. Vegetation on the bank would doubtless have prevented their being discovered.

In 1936 this particular location had been carefully investigated and found to be sound and in excellent condition.

After the washout, the dam was repaired by the City which hired Mr. Overacked of Burlington to carry out this work.

\- - - - - -

June 27, 1942 - Extracts from memo of F. B. Hall to Mr. Wheeler,

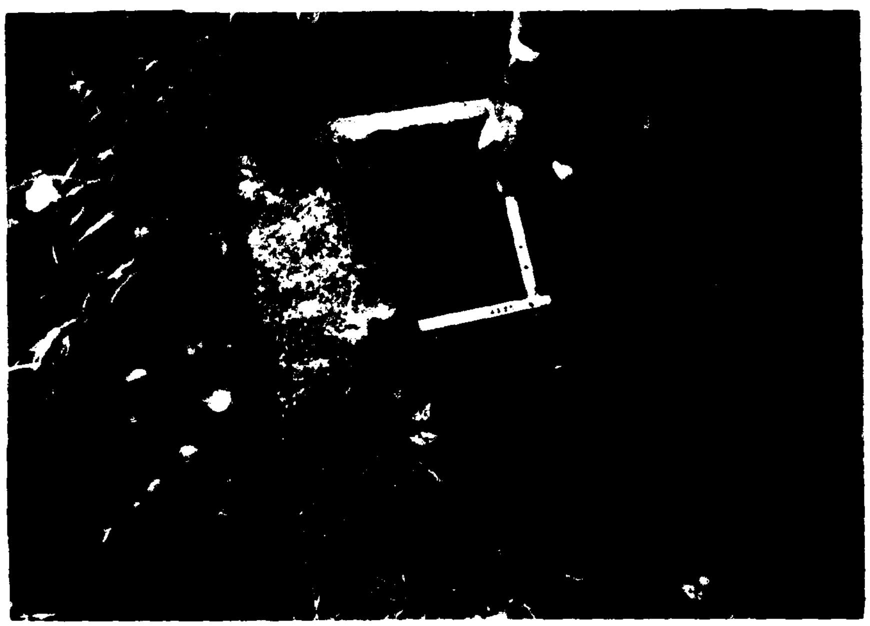
"I inspected the dam very thoroughly. Of course the downstream slope of the main dam is so overgrown with grass and weeds that it is simply impossible to detect any seepage.

"The Mayor wanted to know if we had received a letter from Daniels stating that he wanted to raise the spillway 16-in. He said that Daniels told him he had written us to that effect. I told the Mayor I had not seen such a letter, and if Daniels wrote it, we would like to see a copy."

C-5A Intake structure, control tower, and spillway - 10/24/79

C-5B Control tower and service bridge - 10/24/79

C-6A Damage to concrete at waterline on intermediate pier on left side of service bridge - 10/24/79

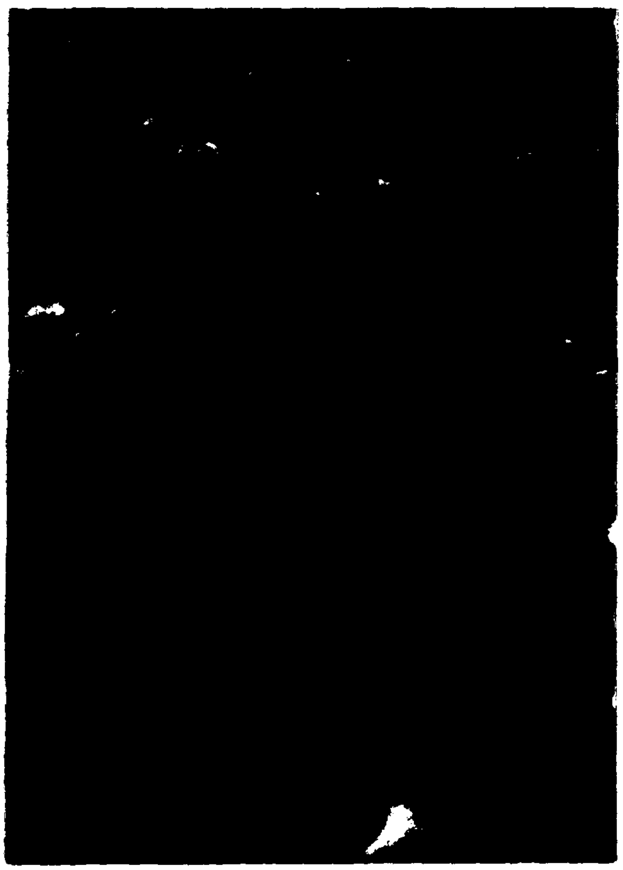


C-6B Crack at abutment of service bridge on dam crest - 10/24/79

C-7A 8-inch diameter water intake piping and valves (high and intermediate levels) looking up inside valve chamber under control tower 10/24/79

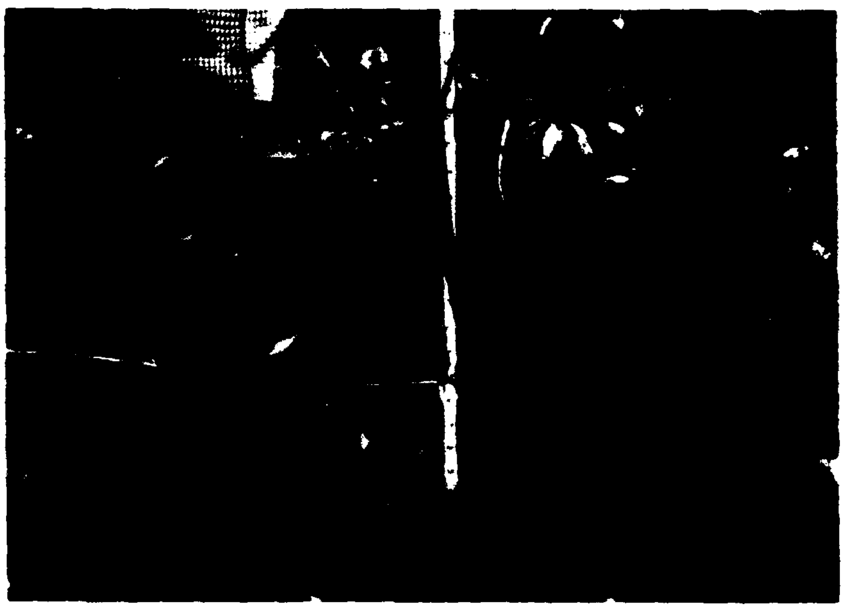
C-7B Bottom of valve chamber under control tower on side toward the spillway. Note 14-inch spur gear blow-off valve in center with the valve chamber drain valves at each side - 10/24/79

C-8A Outlet structure and channel with training walls - 10/24/79



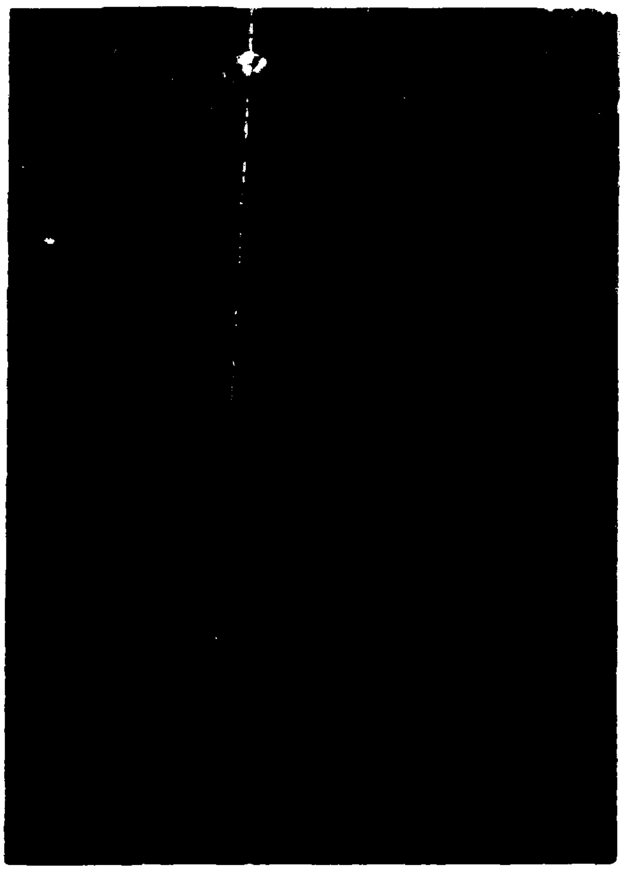
C-8B Cracking and tipping of outlet channel training wall at angle point on right side - 10/24/79

C-9A Discharge channel (beaver pond) looking downstream from outlet structure - 10/24/79

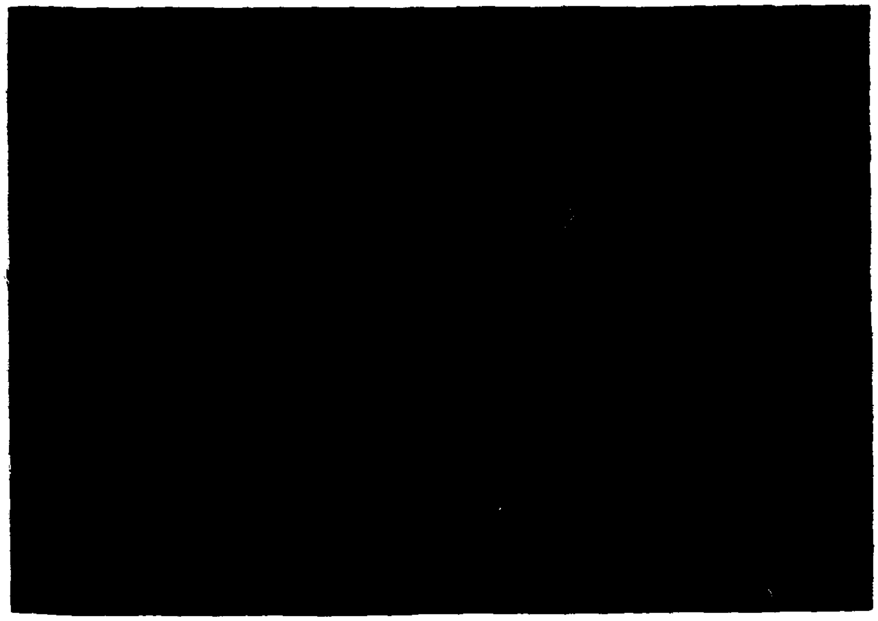


C-9B Seepage at sta 1 + 15L at left abutment contact about 3 feet above toe - 10/24/79

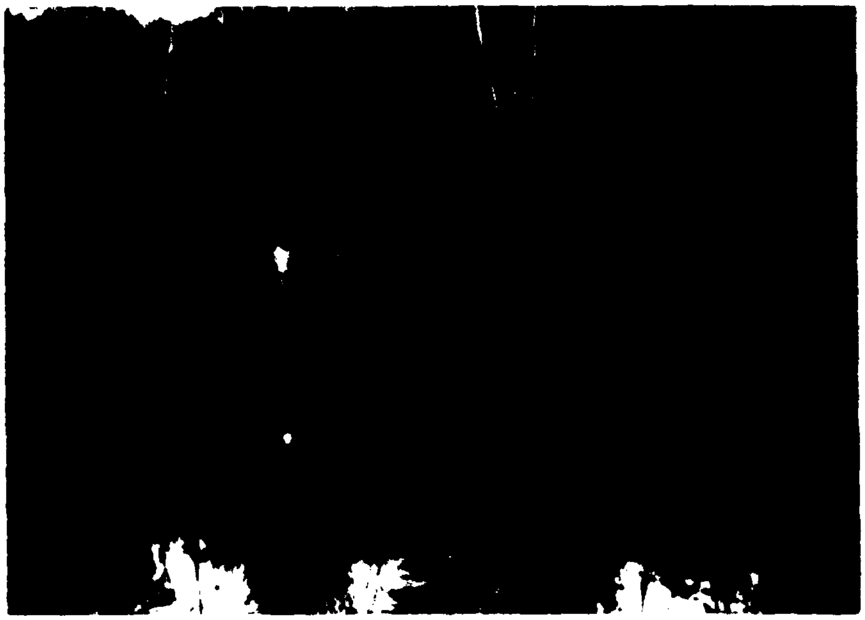
C-10A Sinkhole-type subsidence in dike at sta 2 + 80L on upstream side of corewall - 10/24/79



C-10B Settlement of dike crest on downstream side of corewall from sta 1 + 60L to 2 + 65L. Person is standing on corewall - 10/24/79



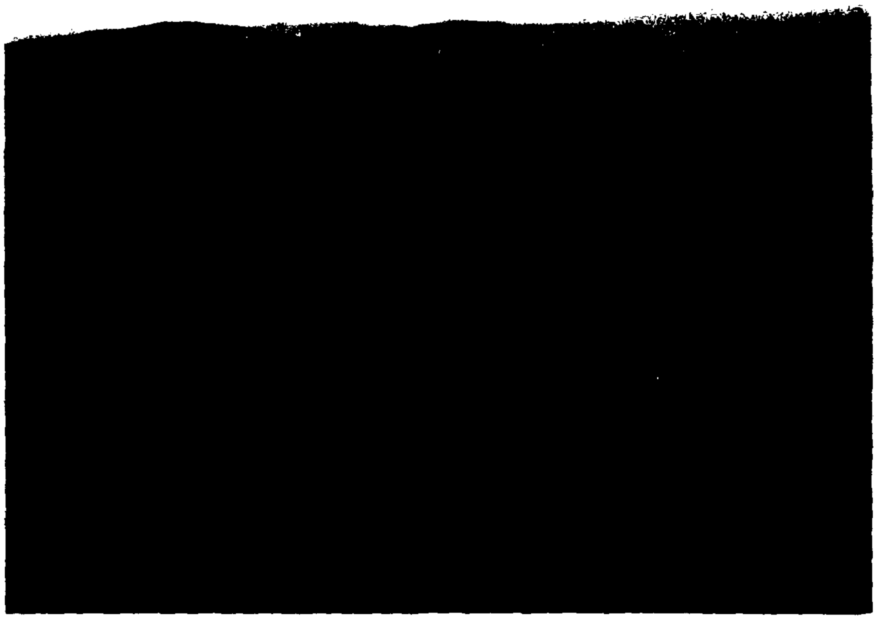
C-11A Aerial view of Rivers Brook diversion dam looking downstream 11/30/79



C-11B Intake structure and spillway of Rivers Brook diversion dam looking from opposite shore - 10/24/79

C-12A Spillway of Rivers Brook diversion dam - 10/24/79

C-12B Aerial overview of reservoir and dam looking downstream
11/30/79



C-13A Aerial view of downstream area looking upstream at reservoir in left background. In right foreground, note outlet stream crossing Plank Road before road bend and access road to the reservoir starting at road bend - 11/30/79

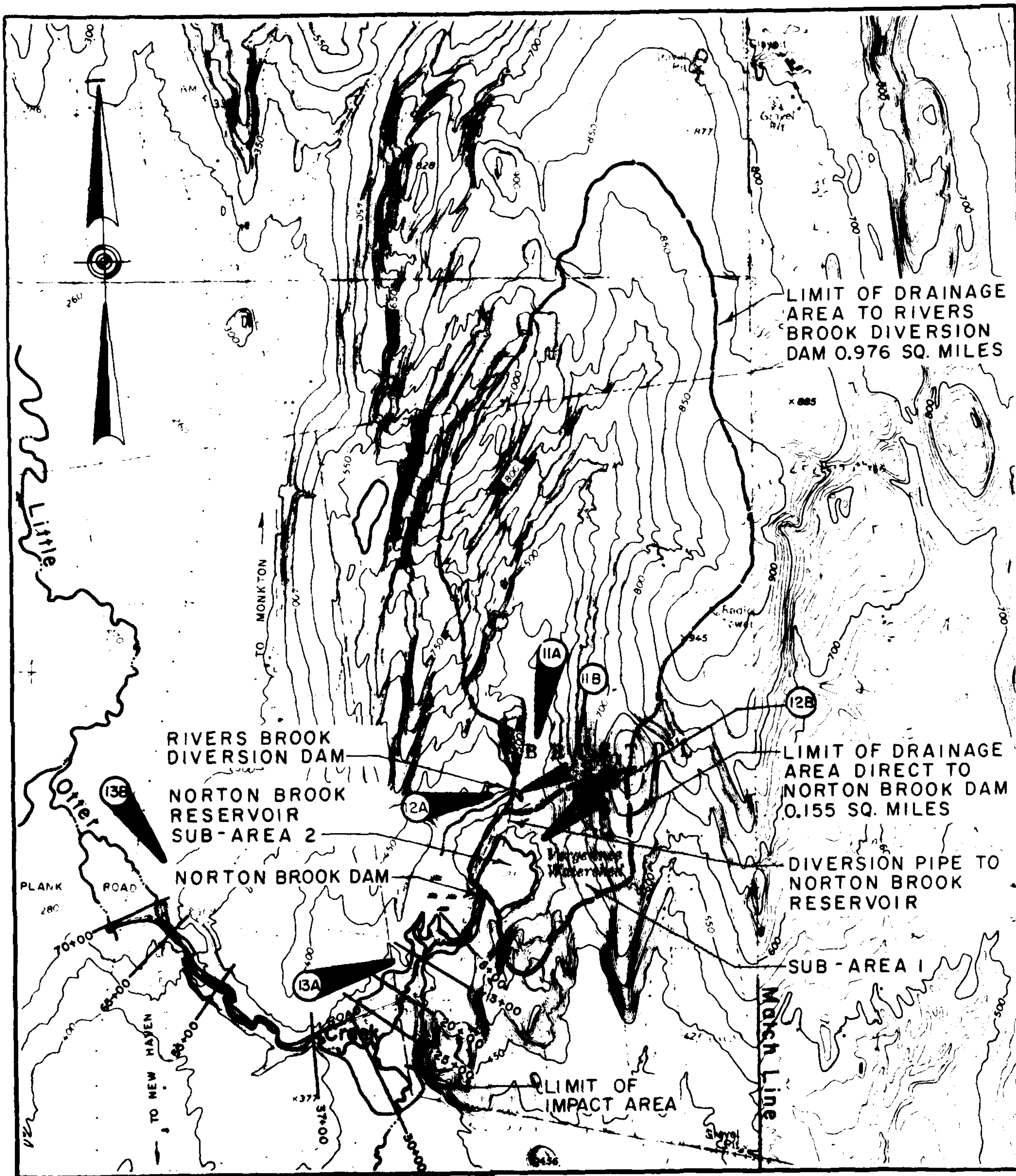
C-13B Aerial view of dowstream hazard at intersection of Plank Road and New Haven/Monkton Road looking upstream. Note channel of Little Otter Creek and adjacent house and trailer - 11/30/79

APPENDIX D

HYDRAULIC AND HYDROLOGIC COMPUTATIONS

TABLE OF CONTENTS

	PAGE
Drainage Area Map	D-1
Elevation - Area - Storage Computations	D-2
Elevation - Area Curve	D-3
Elevation - Storage Curve	D-4
Discharge Computations	
Drop Inlet Computations	D-5
Spillway Outlet Pipe	D-6
Over Dam	D-7
Summary	D-7
Elevation - Discharge Curve	D-8
Inflow From Rivers Brook Diversion Dam	D-9
Drainage Area Data for HEC-1 DB Model	D-10
Overtopping Analysis	
Computer Input	D-11
Computer Output	D-13
Inflow and Outflow Hydrograph Plot	D-16
Dam Failure Analysis	
Plots of Downstream Cross-Sections	D-22
Profile of Downstream Channel	D-26
Dam Breach - Test Flood	
Computer Input	D-27
Computer Output	D-29
Dam Breach - No Flood	
Computer Input	D-39
Computer Output	D-41



SCALE IN FEET

0 1000 2000 4000 6000

DATUM - N.G.V.D. 1929
BASE MAP - 7.5' U.S.G.S. TOPO QUADS.
LEFT SIDE - MONKTON, VT. - 1963
10' CONTOUR INTERVAL
RIGHT SIDE - BRISTOL, VT. - 1963
20' CONTOUR INTERVAL

(11A) - PHOTOS IN APPENDIX C

NORTON BROOK DAM DRAINAGE AREA MAP APPENDIX D-1

GORDON E. AINSWORTH & ASSOCIATES, INC.

Engineers, Surveyors and Planners

20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO. OF
CALCULATED BY ELV DATE 11/26/79
CHECKED BY GPB DATE 12/79
SCALE 21-06-79103

ELEVATION - AREA - STORAGE COMPUTATIONS

RESERVOIR VOLUME: COMPUTED BY METHOD OF CONIC SECTIONS

$$\Delta V_{12} = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$

	ELEVATION (NGVD - ft.)*	AREA (acres)	h (ft.)	ΔV (acre-feet)	V_T (acre-feet)
	355	0.2	6	6.1	0
	361	2.2	10	49.7	6 **
	371	8.4	10	114.0	56 **
SPILLWAY CREST →	381	14.7	4	63.3	170 **
DAM CREST →	385	17.0	15	283.7	233
	400	20.9			517

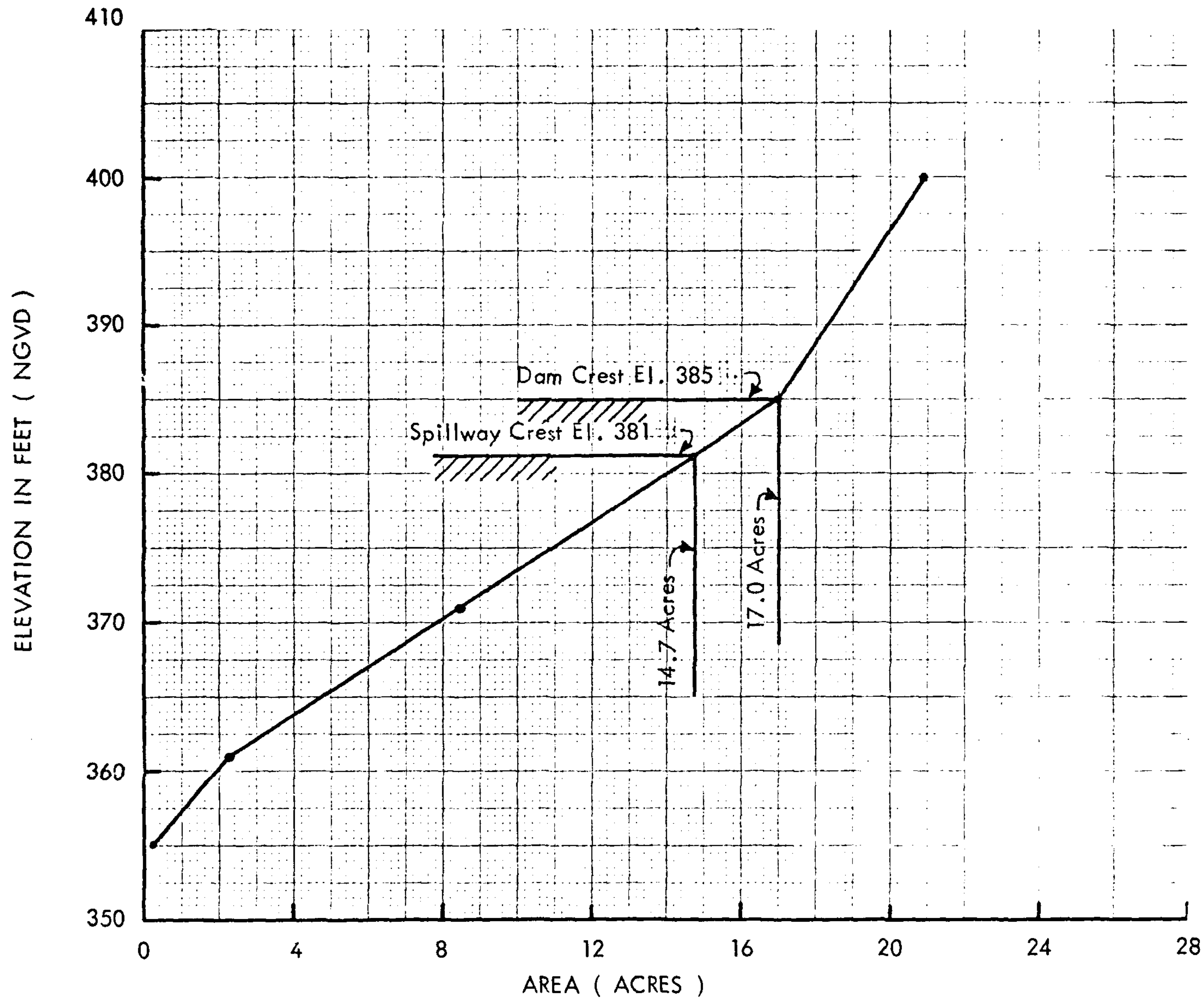
DRAINAGE AREA	AREA (acres)	AREA (square miles)
RESERVOIR SURFACE (SUB-AREA 2) @ NORMAL POOL EL = 381	14.7	0.023
WATERSHED DIRECT TO RESERVOIR (SUB-AREA 1)	84.7	0.132
DRAINAGE AREA TO NORTON BROOK DAM	99.4	0.155
DRAINAGE AREA TO RIVERS BROOK DIVERSION DAM	624.4	0.976

* CONSTRUCTION DRAWING ELEVATION BASE IS APPROXIMATELY 29' HIGHER THAN NGVD ELEVATION.

** CONSTRUCTION DRAWINGS INDICATE $V_T = 5\times10^7$ gal. (153.4 ac-ft) OF STORAGE BETWEEN ELEVATIONS 361 + 381. OUR CALCS. INDICATE V_T = 164 ac-ft BETWEEN THESE ELEVATIONS, OR 7% MORE

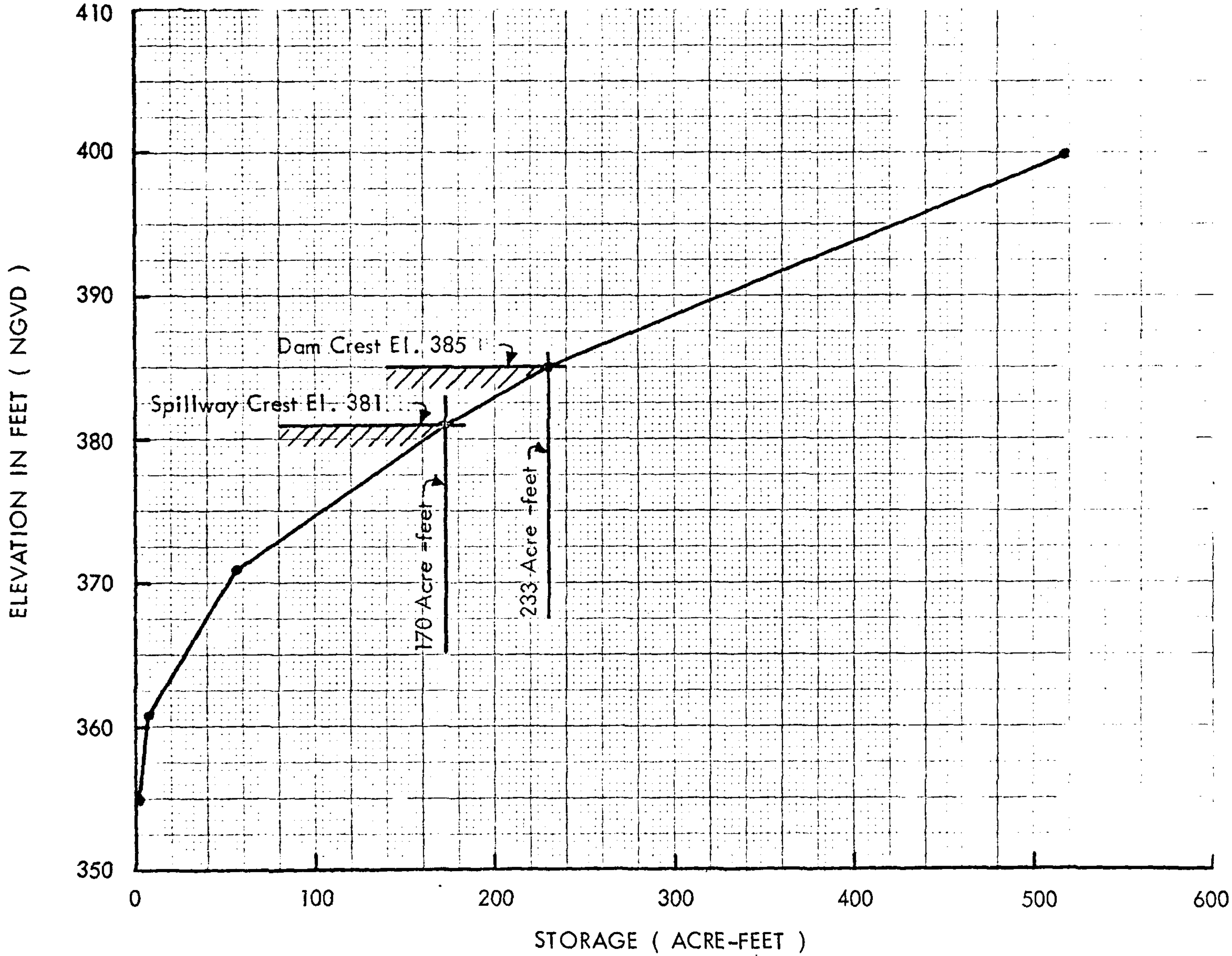
NORTON BROOK DAM, BRISTOL, VERMONT

ELEVATION — AREA



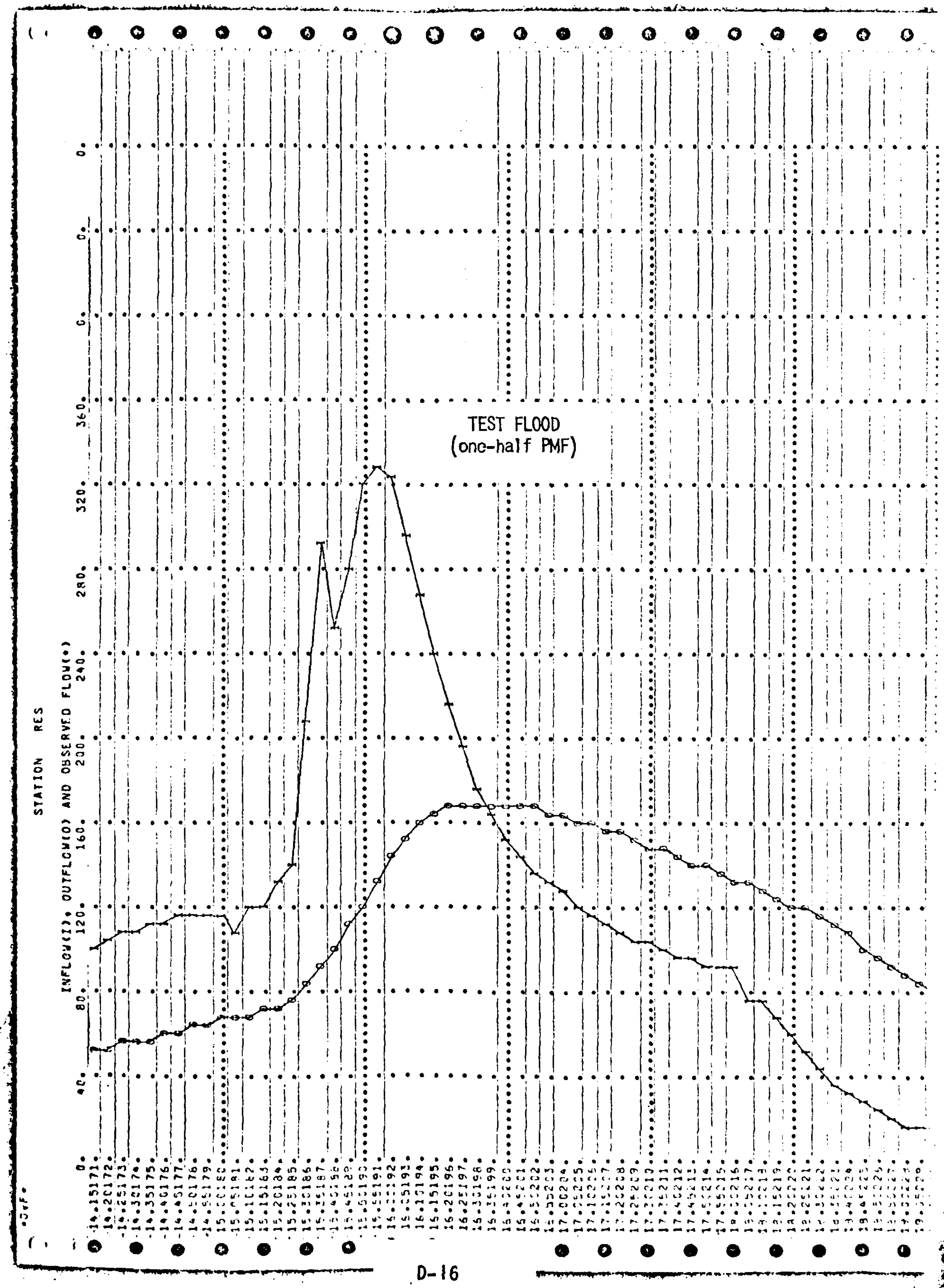
NORTON BROOK DAM, BRISTOL, VERMONT

ELEVATION — STORAGE



```
NORMAL DEPTH CHANNEL ROUTING          STA 37+00
----------------------------

      GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH     SEL
     0.0400   0.0300   0.0400    331.0    350.0     700.  0.00300

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00   350.00    50.00   340.00    94.00   333.00    95.00   331.00   105.00   331.00
         106.00   333.00   140.00   340.00   280.00   350.00

NORMAL DEPTH CHANNEL ROUTING          STA 55+00
----------------------------

      GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH     SEL
     0.0400   0.0300   0.0400    298.0    320.0    1800.  0.01900

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00   320.00    10.00   310.00   160.00   300.00   190.00   298.00   210.00   298.00
         220.00   300.00   250.00   310.00   310.00   320.00

NORMAL DEPTH CHANNEL ROUTING          STA 65+00
----------------------------

      GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH     SEL
     0.0400   0.0300   0.0400    280.0    300.0    1000.  0.01800

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00   300.00   240.00   290.00   270.00   281.00   270.00   280.00   310.00   280.00
         310.00   281.00   350.00   290.00   440.00   300.00

NORMAL DEPTH CHANNEL ROUTING          STA 70+00
----------------------------

      GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH     SEL
     0.0400   0.0300   0.0400    278.0    290.0     500.  0.00400

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00   290.00   170.00   280.00   950.00   279.00   990.00   278.00  1010.00   278.00
        1015.00   279.00  1120.00   280.00  1250.00   290.00

          **********          **********          **********          **********          **********
```

```
*OVN*

NORMAL DEPTH CHANNEL ROUTING          STA 8+00
----------------------------

        QN(1)    QN(2)    QN(3)   ELNVT   ELMAX   RLNTH     SEL
       0.0400   0.0300   0.0400   348.0   365.0    800. 0.00930

        CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  370.00   30.00  360.00   80.00  350.00   95.00  348.00  105.00  348.00
         180.00  350.00  230.00  360.00  255.00  365.00

NORMAL DEPTH CHANNEL ROUTING          STA 13+00
----------------------------

        QN(1)    QN(2)    QN(3)   ELNVT   ELMAX   RLNTH     SEL
       0.0400   0.0300   0.0400   342.0   353.0    500. 0.01200

        CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  360.00   50.00  350.00  144.00  344.00  145.00  342.00  155.00  342.00
         156.00  344.00  300.00  350.00  400.00  353.00

NORMAL DEPTH CHANNEL ROUTING          STA 20+00
----------------------------

        QN(1)    QN(2)    QN(3)   ELNVT   ELMAX   RLNTH     SEL
       0.0400   0.0300   0.0400   338.0   350.0    700. 0.00600

        CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  350.00  100.00  340.00  354.00  339.00  355.00  338.00  365.00  338.00
         366.00  339.00  650.00  340.00 1000.00  350.00

NORMAL DEPTH CHANNEL ROUTING          STA 25+00
----------------------------

        QN(1)    QN(2)    QN(3)   ELNVT   ELMAX   RLNTH     SEL
       0.0400   0.0300   0.0400   336.0   350.0    500. 0.00400

        CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  350.00   50.00  340.00  484.00  337.00  485.00  336.00  495.00  336.00
         496.00  337.00 1060.00  342.00 1250.00  350.00

NORMAL DEPTH CHANNEL ROUTING          STA 30+00
----------------------------

        QN(1)    QN(2)    QN(3)   ELNVT   ELMAX   RLNTH     SEL
       0.0400   0.0300   0.0400   333.0   342.0    500. 0.00600

        CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
```


STATION RES
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0.
40.
80.
120.
160.
200.
240.
280.
320.
360.
TEST FLOOD
(one-half PMF)

```
ELEVATION=      355.      361.      371.      381.      385.      400.

                          CREL    SPWID    COQW    EXPW   ELEVL    COQL   CAREA    EXPL
                         381.0      0.0     0.0     0.0     0.0     0.0     0.0     0.0

                                                       DAM DATA
                                             TOPEL    COQD    EXPD  DAMWID
                                             385.0     3.1     1.5    597.

PEAK OUTFLOW IS      170. AT TIME  16.58 HOURS
```

```
**********          **********          **********          **********          **********

                         SUB-AREA RUNOFF COMPUTATION

SUB-AREA 2 RUNOFF COMPUTATION
                ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE   IAUTO
                 SA-2       0       0       0      0      0       1       0       0

                                 HYDROGRAPH DATA
      IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
          1      -1    0.02    0.00   10.00    0.00   0.000       0       1       0

                                   PRECIP DATA
               SPFE     PMS      R6     R12     R24     R48     R72     R96
               0.00   17.50  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                    LOSS DATA
   LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
       0    0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                                 RECESSION DATA
                STRTQ=   14.00     QRCSN=    0.00     RTIOR= 1.00

             0                   END-OF-PERIOD FLOW
MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q     MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q

                                                                   SUM  18.48  18.48   0.00   7304.
                                                                     ( 469.)( 469.)(   0.)( 206.83)

**********          **********          **********          **********          **********

                              COMBINE HYDROGRAPHS

COMBINING HYDROGRAPHS 1,2
                ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE   IAUTO
                SA-2C       2       0       0      0      0       1       0       0

**********          **********          **********          **********          **********

                               HYDROGRAPH ROUTING

ROUTING FLOW THROUGH RESERVOIR
                ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE   IAUTO
                  RES       1       0       0      2      0       1       0       0
                                  ROUTING DATA
          QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP            LSTR
            0.0   0.000    0.00       1       1       0       0               0

                NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                    1       0       0   0.000   0.000   0.000   -381.      -1

 STAGE     381.00     382.00     383.00     384.00     384.50     385.00     385.50     386.00     387.00     388.00
           389.00

  FLOW       0.00      71.00     196.00     349.00     434.00     540.00     604.00     660.00     759.00     836.00
           836.00

 SURFACE AREA=      0.       2.       8.      15.      17.      21.

    CAPACITY=       0.       6.      56.     170.     233.     517.
```

```
***********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION       JULY 1978
  LAST MODIFICATION  26 FEB 79
***********************************

RUN  DATE?TUE, MAR 04 1980
     TIME?10:08:34

                    NED DAM INSPECTION: DACW33-80-C-0012        NBOT1 OVERTOPPING ANALYSIS
                    VT. 102, NORTON BROOK DAM, 21-06-79103
                    TEST FLOOD  W/ RIVERS BROOK DIVERSION DAM

                                            JOB SPECIFICATION
                    NQ    NHR   NMIN   IDAY    IHR   IMIN  METRC   IPLT   IPRT  NSTAN
                   288      0      5      0      0      0      0      0      4      0
                                       JOPER    NWT  LROPT  TRACE
                                           5      0      0      0

                                 MULTI-PLAN ANALYSES TO BE PERFORMED
                                      NPLAN= 1 NRTIO= 1 LRTIO= 1
                     RTIOS=   0.50

           **********       **********       **********       **********       **********

                                  SUB-AREA RUNOFF COMPUTATION

                  SUB-AREA 1 RUNOFF COMPUTATION
                              ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                               SA-1       0       0       0       0       0       1       0       0

                                            HYDROGRAPH DATA
                   IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                       1       1    0.13    0.00   10.00    0.00   0.000       0       1       0

                                              PRECIP DATA
                              SPFE     PMS      R6     R12     R24     R48     R72     R96
                              0.00   17.50  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                               LOSS DATA
                 LROPT   STRKR   DLTKR   RTIOL   ERAIN   STPKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
                     0    0.00    0.00    1.00    0.00    0.00    1.00    1.00    0.10    0.00    0.00

                                         UNIT HYDROGRAPH DATA
                                     TP=  0.40    CP=0.63    NTA=  0

                                            RECESSION DATA
                              STRTQ=   -4.00    QRCSN=    0.00    RTIOR= 1.00

                  UNIT HYDROGRAPH 27 END-OF-PERIOD ORDINATES, LAG=   0.40 HOURS, CP= 0.63  VCL= 1.00
                    12.       42.       81.      116.      133.      127.      105.       83.       66.       53.
                    42.       33.       26.       21.       17.       13.       11.        8.        7.        5.
                     4.        3.        3.        2.        2.        1.        1.

          0                                   END-OF-PERIOD FLOW
          MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q     MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q

                                                                             SUM  18.48  15.86   2.62   16341.
                                                                                ( 469.)( 403.)( 67.)( 462.73)
```

```
Y1      1
Y6    .04     .03     .04     336     350     500    .004
Y7      0     350      50     340     494     337     485     336     495     336
Y7    496     337    1060     342    1250     350
K       1   30+00                                5       1
K1 CHANNEL ROUTING STA 30+00
Y                                 1       1
Y1      1
Y6    .04     .03     .04     333     342     500    .006
Y7      0     342     100     340    1390     334    1395     333    1405     333
Y7   1410     334    1950     340    2200     350
K       1   37+00                                5       1
K1 CHANNEL ROUTING STA 37+00
Y                                 1       1
Y1      1
Y6    .04     .03     .04     331     350     700    .003
Y7      0     350      50     340      94     333      95     331     105     331
Y7    106     333     140     340     280     350
K       1   55+00                                5       1
K1 CHANNEL ROUTING  STA 55+00
Y                                 1       1
Y1      1
Y6    .04     .03     .04     298     320    1800    .019
Y7      0     320      10     310     180     300     190     298     210     298
Y7    220     300     250     310     310     320
K       1   65+00                                5       1
K1 CHANNEL ROUTING  STA 65+00
Y                                 1       1
Y1      1
Y6    .04     .03     .04     280     300    1000    .018
Y7      0     300     240     290     270     281     270     280     310     280
Y7    310     281     350     290     440     300
K       1   70+00                                5       1
K1 CHANNEL ROUTING STA 70+00
Y                                 1       1
Y1      1
Y6    .04     .03     .04     278     290     500    .004
Y7      0     290     170     280     950     279     990     278    1010     278
Y7   1015     279    1120     280    1250     290
K      99
A
A
A
A
A
```

```
A   NED DAM INSPECTION: DACW33-80-C-0012          NBOT1 OVERTOPPING ANALYSIS
A   VT. 102, NORTON BROOK DAM, 21-06-79103
A   TEST FLOOD  W/ RIVERS BROOK DIVERSION DAM
B    288       0       5                                             4
B1     5
J      1       1       1
J    .50
K      0    SA-1                                    1
K1  SUB-AREA 1 RUNOFF COMPUTATION
M      1       1    .132              10                             1
P           17.5     111     123     132
T                                                 1.0     .10
W     .4    .625
X    -4.               1
K      0    SA-2                                    1
K1  SUB-AREA 2 RUNOFF COMPUTATION
M      1      -1    .023              10                             1
P           17.5     111     123     132
T                                                   0       0
U      1
U1   178
X   14.0               1
K      2    SA-2C                                   1
K1  COMBINING HYDROGRAPHS 1,2
K      1     RES                       2            1
K1  ROUTING FLOW THROUGH RESERVOIR
Y                             1        1
Y1     1                                         -381      -1
Y4   381     382     383     384   384.5     385  385.5     386     387     388
Y4   389
Y5     0      71     196     349     434     540    604     660     759     836
Y5   836
$A    .2     2.2     2.4    14.7    17.0    20.9
$E   355     361     371     381     385     400
$$   381
$D   395   3.087     1.5     597
K      1    8+00                         5          1
K1  CHANNEL ROUTING STA 8+00
Y                             1        1
Y1     1
Y6   .04     .03     .04     348     365     800   .009
Y7     0     370      30     360      80     350     95     348     105     348
Y7   180     350     230     360     255     365
K      1   13+00                         5          1
K1  CHANNEL ROUTING STA 13+00
Y                             1        1
Y1     1
Y6   .04     .03     .04     342     353     500   .012
Y7     0     360      50     350     144     344    145     342     155     342
Y7   156     344     300     350     400     353
K      1   20+00                         5          1
K1  CHANNEL ROUTING STA 20+00
Y                             1        1
Y1     1
Y6   .04     .03     .04     338     350     700   .006
Y7     0     350     100     340     354     339    355     338     365     338
Y7   366     339     660     340    1000     350
K      1   25+00                         5          1
K1  CHANNEL ROUTING STA 25+00
Y                             1        1
```

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 11/29/79
CHECKED BY JPB DATE 12/79
SCALE 21-06-79103

DRAINAGE AREA DATA FOR HEC-1DB MODEL

SUB-AREA 1: AREA TRIBUTARY DIRECTLY TO RESERVOIR
AREA = 0.132 SQUARE MILES

LOSS RATES: 1.0" - INITIALLY
0.1"/HOUR - CONSTANT LOSS RATE

UNIT HYDROGRAPH PARAMETERS: USE SNYDER METHOD

A = DRAINAGE AREA = 0.132 SQUARE MILES

L = LENGTH OF MAIN WATERCOURSE TO UPSTREAM LIMIT OF DRAINAGE AREA = 0.34 MILES

L_{CA} = LENGTH ALONG MAIN WATERCOURSE TO POINT OPPOSITE THE CENTROID OF THE DRAINAGE AREA = 0.057 MILES

C_t = SNYDER'S BASIN COEFFICIENT = 2.0 ASSUMED AVERAGE

C_p = SNYDER'S PEAKING COEFFICIENT = 0.625 ASSUMED AVERAGE

t_p = STANDARD LAG IN HOURS = $C_t (L L_{CA})^{0.3}$ = 0.61 HOURS

ALTERNATE $t_p = \dfrac{\text{flow length}}{\text{flow velocity}} = \dfrac{1800'}{2 \text{ fps}} = 0.25$ HOURS

$\therefore$ USE $t_p = 0.4$ HOURS (BETWEEN CALCULATED VALUES)

SUB-AREA 2: RESERVOIR SURFACE, AREA = 0.023 SQUARE MILES (14.7 ACRES)

LOSS RATES: NONE BECAUSE RAINFALL ≈ RUNOFF FOR WATER SURFACE

UNIT HYDROGRAPH PARAMETERS:

FOR U.H. w/ 5 MINUTE DURATION & 1" RAIN

$$\bar{Q} = \frac{A(1")}{t} = \frac{14.7 \text{ acres } (1")}{5 \text{ minutes}} \left(\frac{43560 \text{ sq. ft.}}{1 \text{ acre}}\right)\left(\frac{1 \text{ FT}}{12 \text{ inches}}\right)\left(\frac{1 \text{ minute}}{60 \text{ seconds}}\right)$$

$\bar{Q}$ = 178 cfs (SINCE NO LOSS RATE)

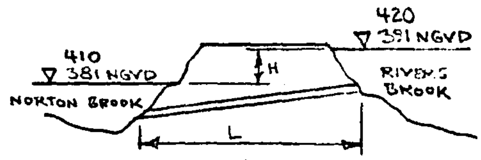
above toe – 10/24/79

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY ELV DATE 11/29/79
CHECKED BY [illegible] DATE 12/79
SCALE 21-06-79103

INFLOW FROM RIVERS BROOK DIVERSION DAM

A 15" DIA CONCRETE PIPE DRAINS RIVERS BROOK DIVERSION DAM TO NORTON BROOK RESERVOIR. ASSUMING THAT THIS PIPE IS FULLY OPEN ITS MAXIMUM CAPACITY CAN BE DETERMINED WITH THE FOLLOWING DATA:



$L = 920'$
$D = 1.25'$ (15")
$n = .013$ (ASSUMED SINCE PIPE IS CONC.)
$S_{ws} = \frac{H}{L} = 0.0109$ (IF WATER SURFACE IS ASSUMED TO BE AT SPILLWAY CRESTS OF BOTH RIVERS BROOK DIVERSION DAM + NORTON BROOK DAM AT ALL TIMES)
$H = 10'$

FOR INLET CONTROL: (DERIVED FROM NOMOGRAPH DATA IN REFERENCE 17 FOR CIRCULAR CONCRETE PIPE WITH POOREST ENTRANCE CONDITION – TYPE 1, SQUARE EDGE W/ HEADWALL)

$$Q = 8.52(H_w)^{.409} = 8.52(10)^{.409}$$
$$Q = 21.8 \text{ cfs}.$$

FOR OUTLET CONTROL: (DERIVED FROM APPLICATION OF BERNOUILLI'S EQUATION USING MANNING'S EQUATION FOR FRICTION LOSS)

K_{en} ASSUMED TO BE 0.5
K_{ex} ASSUMED TO BE 1.0

$$Q = \left[\frac{S_{ws}}{\frac{K_{en}+K_{ex}}{2gA^2L} + \frac{n^2}{2.21A^2R^{4/3}}}\right]^{1/2}$$

$$A = \frac{\pi D^2}{4} = 1.23 \qquad R = \frac{A}{P} = 0.313$$

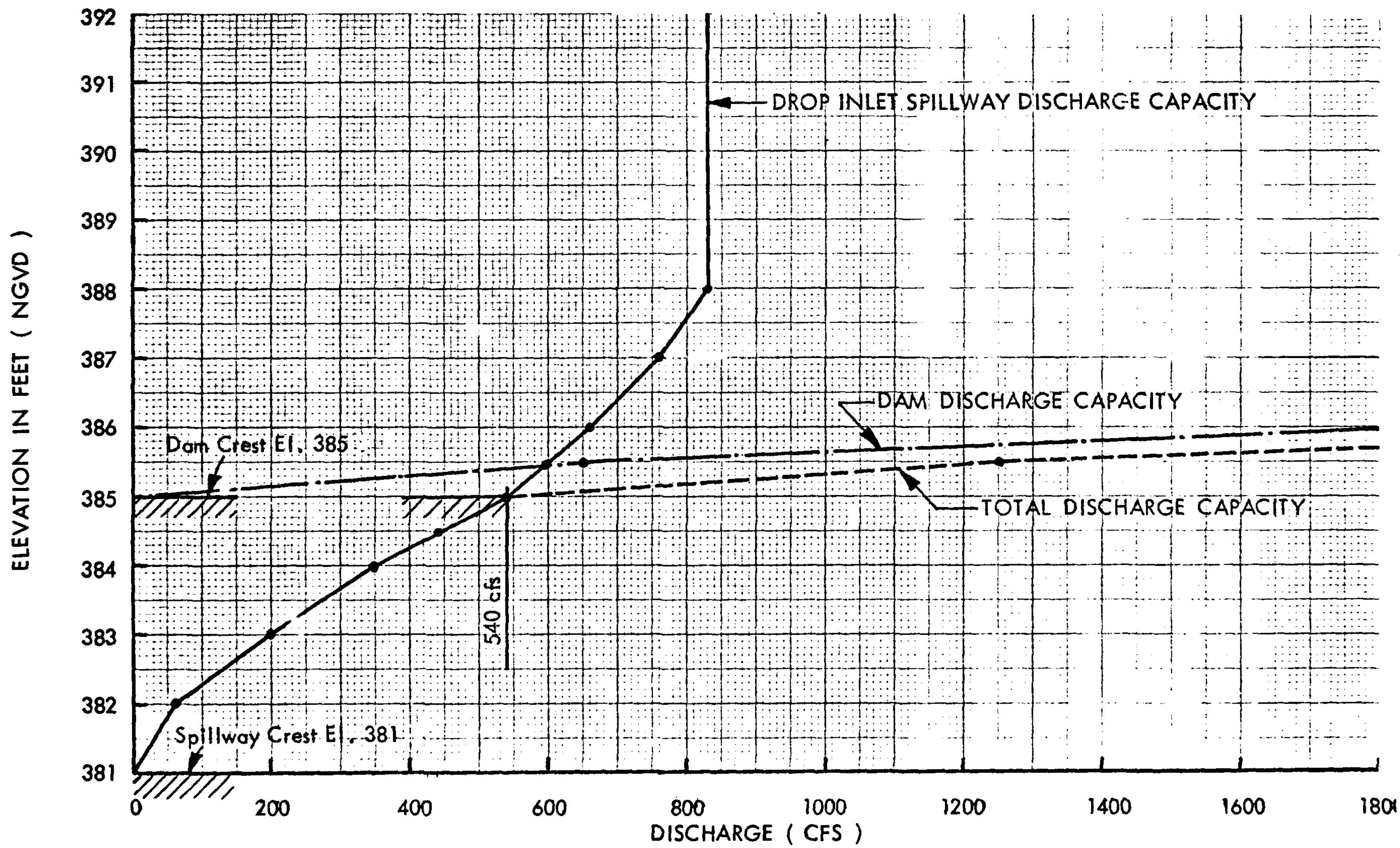
$$Q = 6.5 \approx 7 \text{ cfs}$$

∴ PIPE FROM RIVERS BROOK DIVERSION DAM IS OUTLET CONTROLED.

INFLOW FROM RIVERS BROOK DIVERSION DAM $= \underline{7 \text{ cfs}}$

NORTON BROOK DAM, BRISTOL, VERMONT

ELEVATION — DISCHARGE



GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY CLV DATE 12/6/79
CHECKED BY JPB DATE 12/79
SCALE 21-06-79103

DISCHARGE COMPUTATIONS

DAM APPURTENANCE	ELEVATION (NGVD)	SIZE
DROP INLET SPILLWAY	CREST EL. = 381	22' TOTAL LENGTH w/ 3 OPENINGS ALL 3.5' HIGH
DAM + DIKE	CREST EL. = 385 (LEVEL)	597' CREST LENGTH
OUTLET PIPES:		
WATER SUPPLY MAIN INLET PORTS ①	INV. EL. ≈ 361	8" DIA CIP
②	INV. EL. ≈ 369.5	8" DIA CIP
③	INV. EL. ≈ 378	8" DIA CIP
DRAIN PIPE	INV. EL. ≈ 358	14" DIA CIP

FOR FLOW OVER DAM: $Q_{DAM} = 3.087 L H^{3/2}$ (FORMULA FOR CRITICAL FLOW OVER A BROAD-CRESTED WEIR) REFERENCE 9

ELEVATION (NGVD)	$H_{SPILLWAY}$ (feet)	H_{DAM} (feet)	$Q_{SPILLWAY}$ (cfs)	Q_{DAM} (cfs)	$Q_{OUTLET\ PIPES}$ (cfs)	Q_T (cfs)
381	0	0	0	0	0	0
382	1	0	71	0	ALL OUTLETS ASSUMED CLOSED	71
383	2	0	196	0		196
384	3	0	349	0		349
384.5	3.5	0	434	0		434
385	4	0	540	0		540
385.5	4.5	.5	604	652		1256
386	5	1	660	1843		2503
387	6	2	759	5213		5972
388	7	3	836	9576		10412
389	8	4	836	14744		15580
390	9	5	836	20605	0	21441

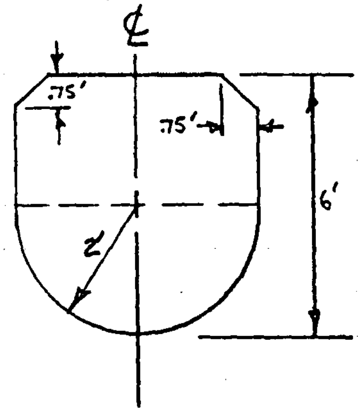
GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB: NORTON BROOK DAM
SHEET NO. ____ OF ____
CALCULATED BY: ELV DATE: 11/27/79
CHECKED BY: [illegible] DATE: 12/79
SCALE: 21-06-79103

DISCHARGE COMPUTATIONS

CAPACITY OF SPILLWAY OUTLET PIPE - FOUND USING MANNING'S EQUATION FOR FULL PIPE FLOW

$$Q = \frac{1.486}{n} \frac{A^{5/3}}{P^{2/3}} S^{1/2}$$ (MANNING'S EQUATION)



$$P = \tfrac{1}{2}[2\pi(2)] + 2(2-.75) + 2(.75\sqrt{2}) + [8-2(.75)]$$
$$P = 17.40'$$

$$A = \tfrac{1}{2}\pi(2)^2 + 4(6) - 2(\tfrac{1}{2})(.75)(.75)$$
$$A = 29.7\ \text{ft}^2$$

$$S = 3'/101' = .0297\ \text{ft/ft}$$

$n = .013$ (ASSUMED MANNING'S n FOR CONC. CONDUIT)

$$Q = \frac{1.486}{.013} \frac{(29.7)^{5/3}}{(17.40)^{2/3}} (.0297)^{1/2}$$

$Q = 836$ cfs (MAX FLOW THROUGH OUTLET CONDUIT FLOWING FULL)

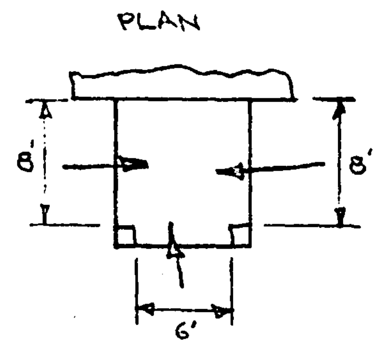
$\therefore$ **$Q = 840^{\pm}$ cfs**

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 11/27/79
CHECKED BY QPB DATE 12/79
SCALE 21-06-79103

DISCHARGE COMPUTATIONS

DROP INLET SPILLWAY CAPACITY



SPILLWAY CONSISTS OF:

1 - 6' WIDE x 3.5' HIGH SHARP-CRESTED WEIR
2 - 8' WIDE x 3.5' HIGH SHARP-CRESTED WEIRS

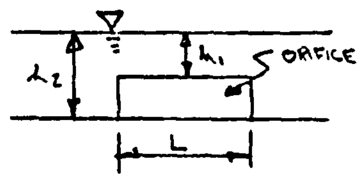
FOR FLOW 0' ≤ 3.5' DEEP WEIR FLOW ASSUMED:

$$Q = 3.33\,(L - 0.2H)\,H^{1.5} \quad \text{(REFERENCE 16)}$$

WITH: L = length, 0.2H = end losses, + SHARP-CRESTED RECTANGULAR WEIR

FOR FLOW > 3.5' ORIFICE FLOW ASSUMED:

$$Q = \frac{2}{3} L C \sqrt{2g}\left(h_2^{3/2} - h_1^{3/2}\right) \quad \text{(REFERENCE 9)}$$



WITH: C = ORIFICE COEF. = 0.6

WATER DEPTH (Feet)	6' WIDTH WEIR Q (cfs)	6' WIDTH ORIFICE Q (cfs)	6' WIDTH Q (cfs)	8' WIDTH WEIR Q (cfs)	8' WIDTH ORIFICE Q (cfs)	8' WIDTH Q (cfs)	TOTAL WIDTH (1-6' + 2-8') Q_T (cfs)
0	0		0	0		0	0
1	19.3		19.3	26.0		26.0	71
2	52.7		52.7	71.6		71.6	196
3	93.4		93.4	128.0		128.0	349
3.5	115.6	126.1	115.6	159.2	168.1	159.2	434
4		147.2	147.2		196.4	196.4	540
4.5		164.6	164.6		219.5	219.5	604
5		179.9	179.9		239.9	239.9	660
6		206.9	206.9		275.9	275.9	759 ← *
7		230.6	230.6		307.4	307.4	845
8		251.9	251.9		335.9	335.9	924

* MAX FLOW POSSIBLE THROUGH OUTLET CONDUIT IS 840± cfs

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 0.50
HYDROGRAPH AT	SA-1	0.13	1	302.
		(0.34)		(8.54)(
HYDROGRAPH AT	SA-2	0.02	1	154.
		(0.06)		(4.37)(
2 COMBINED	SA-2C	0.15	1	330.
		(0.40)		(9.33)(
ROUTED TO	RES	0.15	1	170.
		(0.40)		(4.81)(
ROUTED TO	8+00	0.15	1	170.
		(0.40)		(4.81)(
ROUTED TO	13+00	0.15	1	170.
		(0.40)		(4.81)(
ROUTED TO	20+00	0.15	1	169.
		(0.40)		(4.79)(
ROUTED TO	25+00	0.15	1	169.
		(0.40)		(4.78)(
ROUTED TO	30+00	0.15	1	169.
		(0.40)		(4.78)(
ROUTED TO	37+00	0.15	1	169.
		(0.40)		(4.78)(
ROUTED TO	55+00	0.15	1	168.
		(0.40)		(4.77)(
ROUTED TO	65+00	0.15	1	168.
		(0.40)		(4.76)(
ROUTED TO	70+00	0.15	1	168.
		(0.40)		(4.76)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	381.00	381.00	385.00
STORAGE	170.	170.	233.
OUTFLOW	0.	0.	540.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.50	382.79	0.00	197.	170.	0.00	16.58	0.00

PLAN 1 STATION 8+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	170.	349.1	16.58

PLAN 1 STATIO 13+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	170.	344.0	16.58

PLAN 1 STATION 20+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	169.	339.4	16.75

PLAN 1 STATION 25+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	169.	337.6	16.83

PLAN 1 STATION 30+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	169.	334.5	16.83

PLAN 1 STATION 37+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	169.	334.0	16.92

PLAN 1 STATION 55+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	168.	299.0	17.00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	168.	280.6	17.00

PLAN 1 STATION 70+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	168.	279.2	17.08

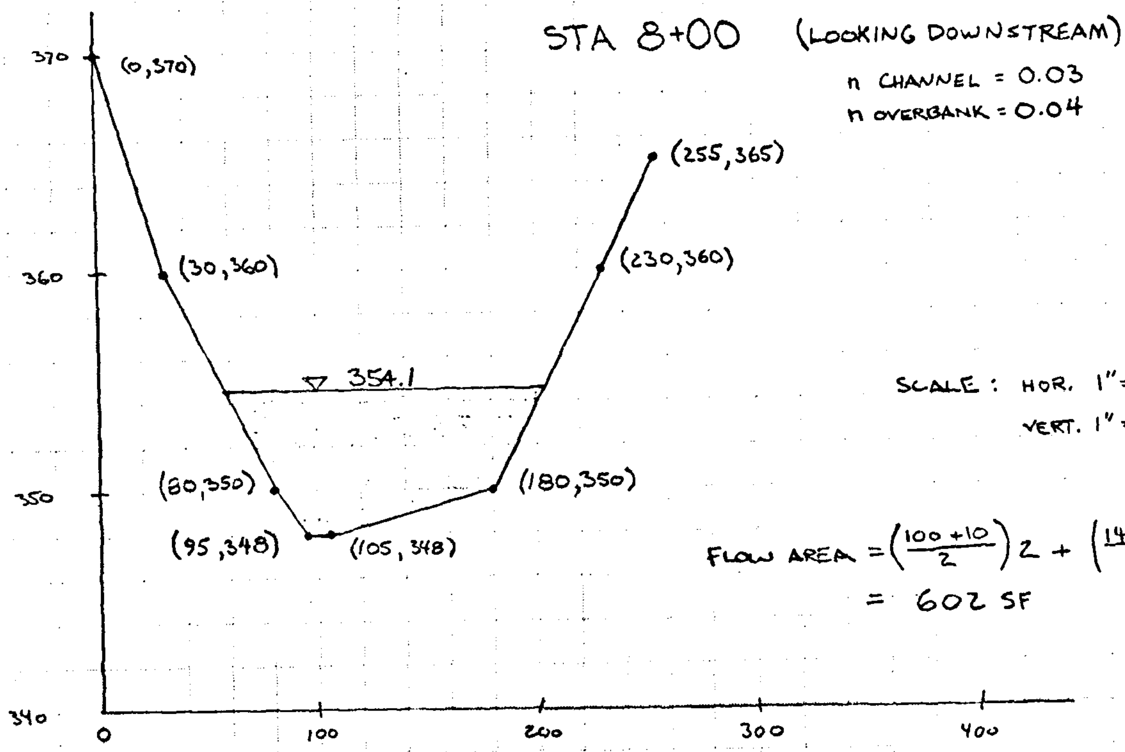
GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB: NORTON BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY: CLV DATE: 12/21/79
CHECKED BY: [illegible] DATE: 1/80
SCALE ______

STA 8+00 (LOOKING DOWNSTREAM)

n CHANNEL = 0.03
n OVERBANK = 0.04

SCALE: HOR. 1" = 80'
VERT. 1" = 8'



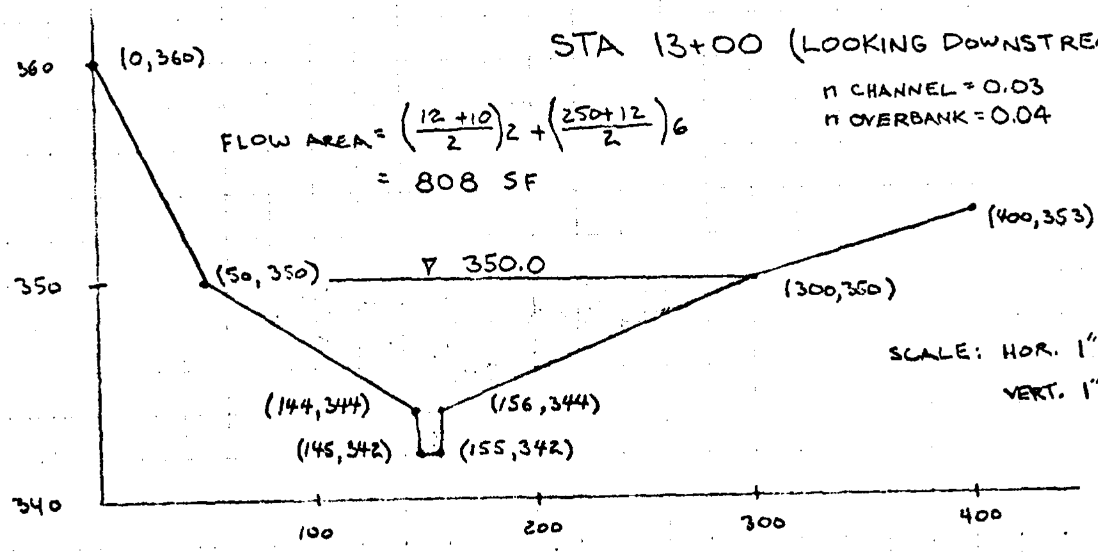
FLOW AREA $= \left(\frac{100+10}{2}\right)2 + \left(\frac{140+100}{2}\right)4.1$
$= 602$ SF

STA 13+00 (LOOKING DOWNSTREAM)

n CHANNEL = 0.03
n OVERBANK = 0.04

FLOW AREA $= \left(\frac{12+10}{2}\right)2 + \left(\frac{250+12}{2}\right)6$
$= 808$ SF

SCALE: HOR. 1" = 80'
VERT. 1" = 8'



FORM 204 Available from NEBS INC. Townsend, Mass 01470

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

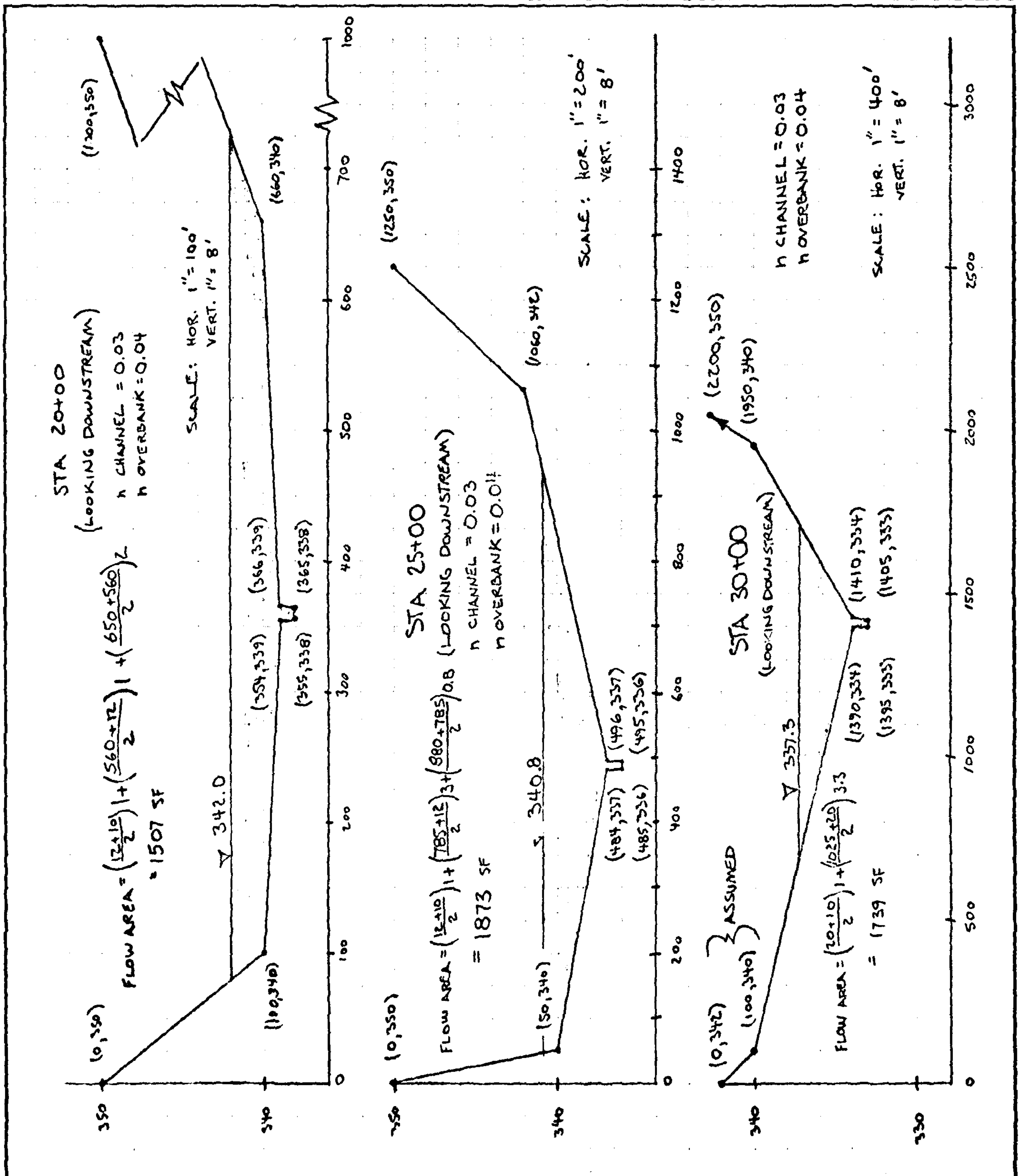
JOB NORTON BROOK DAM

SHEET NO. ______ OF ______

CALCULATED BY ELV DATE 12/27/79

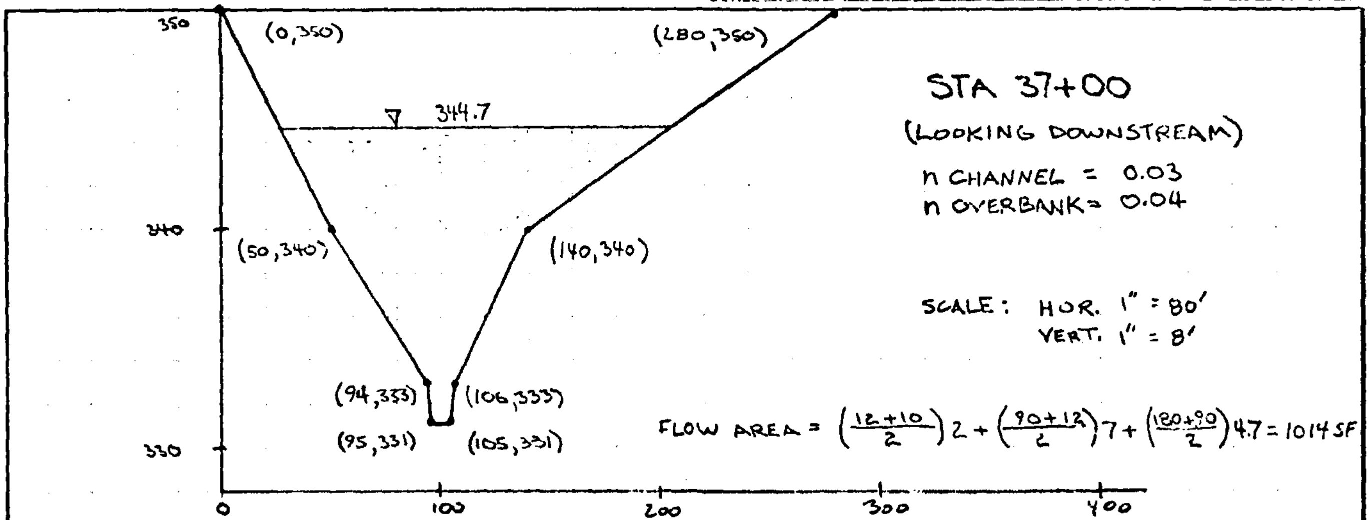
CHECKED BY JPO DATE 1/80

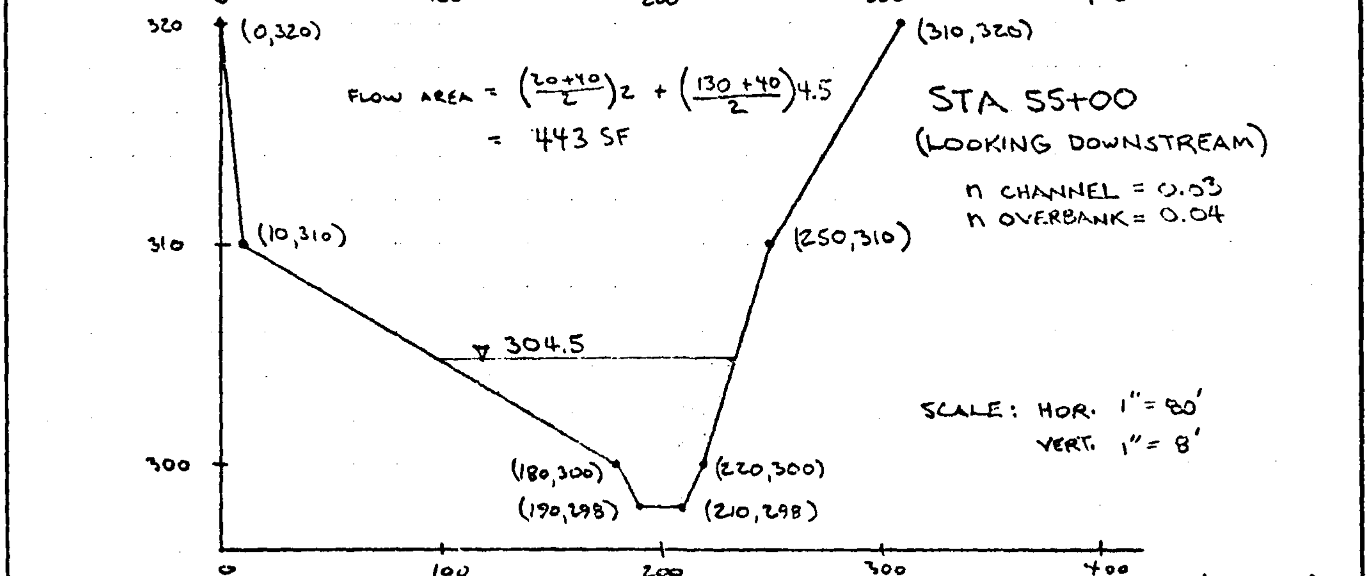
SCALE ______

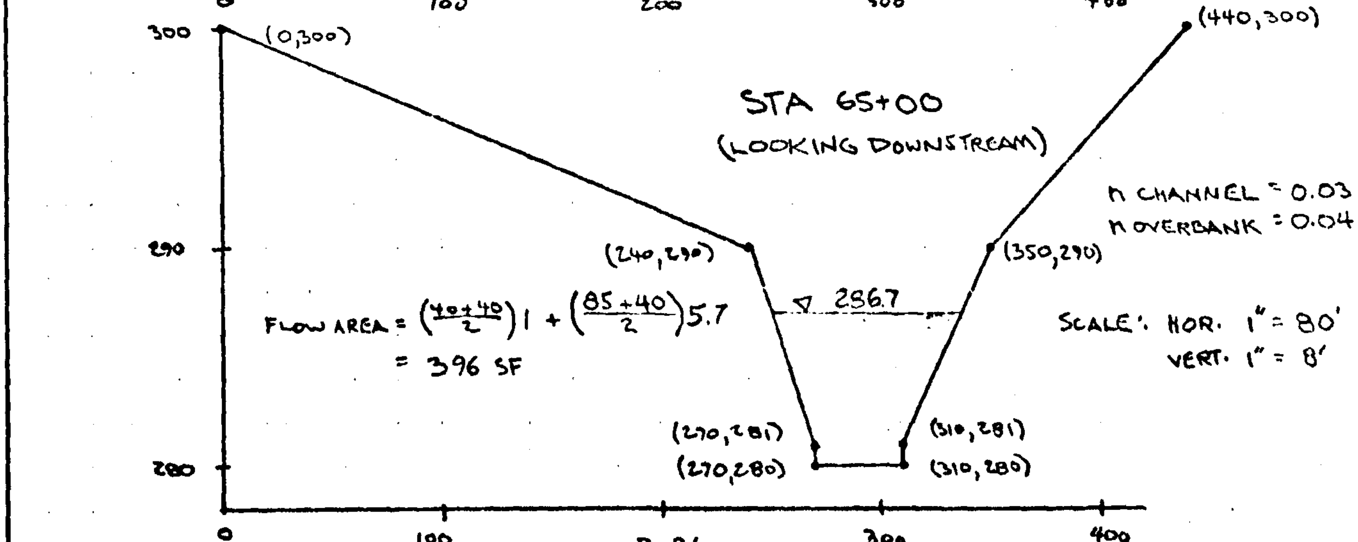


GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO ______ OF ______
CALCULATED BY ELV DATE 12/27/79
CHECKED BY JPB DATE 1/80
SCALE ______

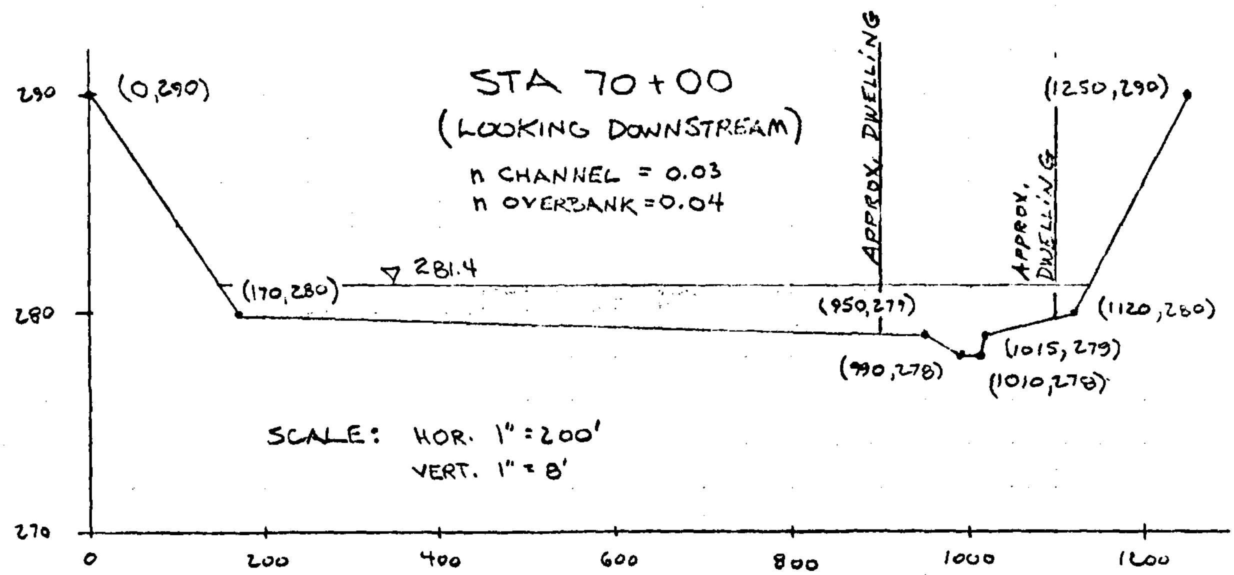






GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB NORTON BROOK DAM
SHEET NO. ________ OF ________
CALCULATED BY ELV DATE 12/27/79
CHECKED BY JMB DATE 1/80
SCALE ________



$$\text{FLOW AREA} = \left(\frac{65+20}{2}\right)1 + \left(\frac{950+65}{2}\right)1 + \left(\frac{990+950}{2}\right)1.4$$

$$= 1909 \text{ SF}$$

G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

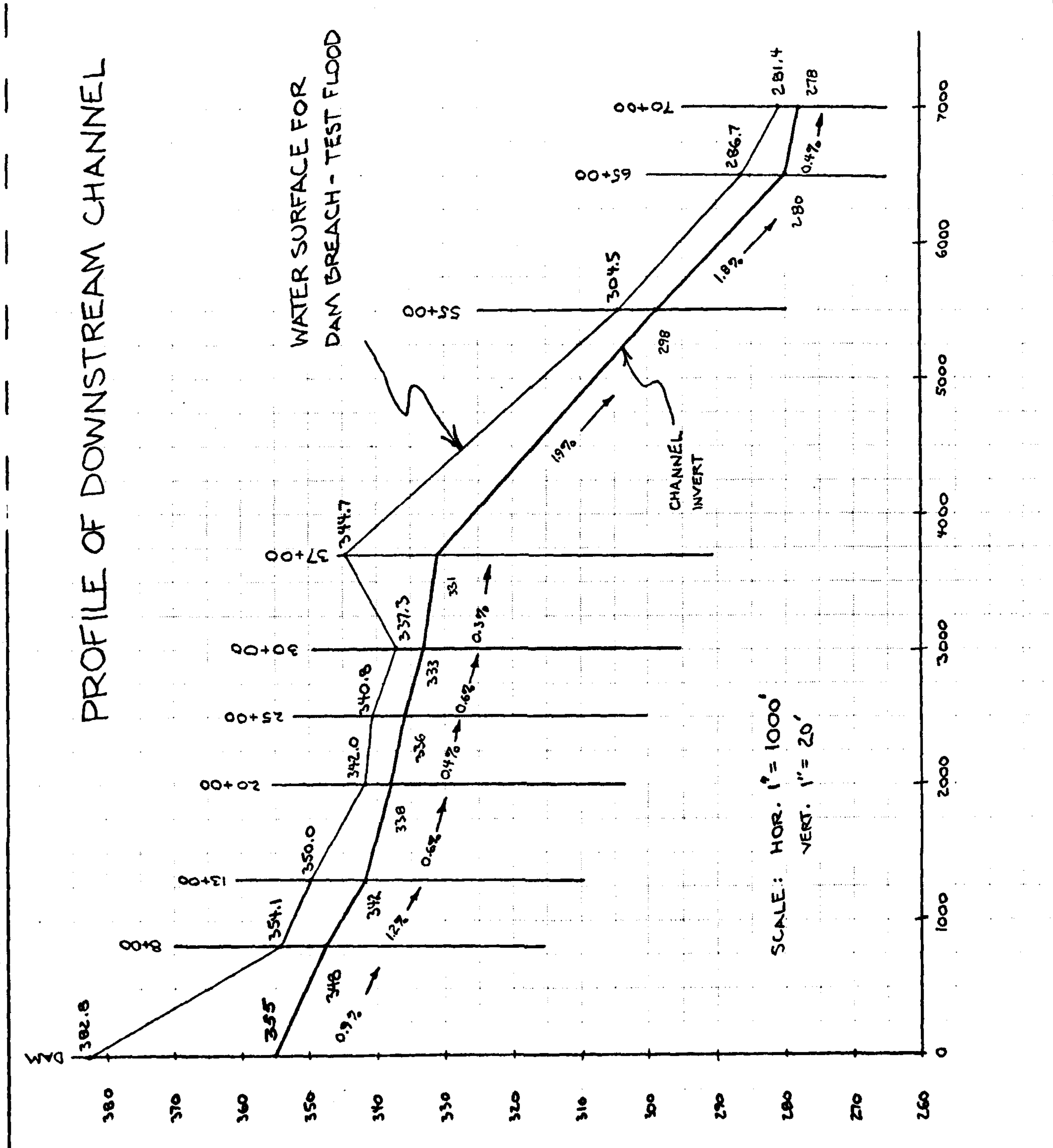
JOB NORTON BROOK DAM

SHEET NO. ______ OF ______

CALCULATED BY ELV DATE 3/11/80

CHECKED BY [illegible] DATE 3/80

SCALE ______



```
A   NED DAM INSPECTION: DACW33-80-C-0012        NBDB1 DAMBREAK
A   VT. 102, NORTON BROOK DAM, 21-06-79103
A   TEST FLOOD   W/ RIVERS BROOK DIVERSION DAM
B     298        0       5                                                       9
B1      5
J       1        1       1
J1    .50
K       0    SA-1                                      1
K1 SUB-AREA 1 RUNOFF COMPUTATION
M       1        1    .132            10                                         1
P             17.5     111     123   132
T                                                     1.0     .10
W      .9     .625
X     -4.                1
K       0    SA-2                                      1
K1 SUB-AREA 2 RUNOFF COMPUTATION
M       1       -1    .023            10                                         1
P             17.5     111     123   132
T                                                      0        0
U       1
U1    178
X    14.0        0       1
K       2    SA-2C                                     1
K1 COMBINING HYDROGRAPHS 1,2
K       1     RES                                      1
K1 ROUTING FLOW THROUGH RESERVOIR
Y                                1       1
Y1      1                                             -381      -1
Y4    381      382     383     384   384.5     385   385.5     386     387     388
Y4    389
Y5      0       71     196     349     434     540     604     660     759     836
Y5    634
$A    2.2      2.2     9.4    14.7    17.0    20.9
$E    355      361     371     381     385     400
$$    381
$D    385    3.087     1.5     597
$B     50       .5     355      .5     381  382.78
K       1     8+00                             5       1
K1 CHANNEL ROUTING STA 8+00
Y                                1       1
Y1      1
Y6    .04      .03     .04     348     365     800    .009
Y7      0      370      30     360      80     350      95     348     105     348
Y7    190      350     230     360     255     365
K       1    13+00                             5       1
K1 CHANNEL ROUTING STA 13+00
Y                                1       1
Y1      1
Y6    .04      .03     .04     342     353     500    .012
Y7      0      340      50     350     144     344     145     342     155     342
Y7    156      344     200     350     400     353
K       1    20+00                             5       1
K1 CHANNEL ROUTING STA 20+00
Y                                1       1
Y1      1
Y6    .04      .03     .04     338     350     700    .006
Y7      0      350     100     340     354     339     355     338     365     339
Y7    366      339     560     340    1000     350
K       1    25+00                             5       1
K1 CHANNEL ROUTING STA 25+00
```

```
Y                                  1        1
Y1      1
Y6    .04     .03     .04     336     350     500    .004
Y7      0     350      50     340     484     337     485     336     495     336
Y7    495     337    1060     342    1250     350
K       1   30+00                                   5       1
K1 CHANNEL ROUTING STA 30+00
Y                                  1        1
Y1      1
Y6    .04     .03     .04     333     342     500    .006
Y7      0     342     100     340    1390     334    1395     333    1405     333
Y7   1410     334     195     340    2200     350
K       1   37+00                                   5       1
K1 CHANNEL ROUTING STA 37+00
Y                                  1        1
Y1      1
Y6    .04     .03     .04     331     350     700    .003
Y7      0     350      50     340      94     333      95     331     105     331
Y7    106     333     140     340     280     350
K       1   55+00                                   5       1
K1 CHANNEL ROUTING  STA 55+00
Y                                  1        1
Y1      1
Y6    .04     .03     .04     298     320    1800    .019
Y7      0     220      10     310     180     300     190     298     210     298
Y7    220     300     250     310     310     320
K       1   65+00                                   5       1
K1 CHANNEL ROUTING  STA 65+00
Y                                  1        1
Y1      1
Y6    .04     .03     .04     280     300    1000    .018
Y7      0     300     240     290     270     281     270     280     310     280
Y7    310     281     350     290     440     300
K       1   70+00                                   5       1
K1 CHANNEL ROUTING STA 70+00
Y                                  1        1
Y1      1
Y6    .04     .03     .04     278     290     500    .004
Y7      0     290     170     280     950     279     990     278    1010     278
Y7   1015     279    1120     280    1250     290
K      99
A
A
A
A
A
```

```
**************************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
**************************************

RUN  DATE?MON, MAR 10 1980
     TIME?11:45:26

                    NED DAM INSPECTION: DACW33-80-C-0012        NBDE1 DAMBFEAK
                    VT. 102, NORIGN BROOK DAM, 21-06-79103
                    TEST FLOOD   W/ RIVERS BROOK DIVERSION DAM

                                        JOB SPECIFICATION
                    NQ     NHR    NMIN    IDAY    IHR    IMIN   METRC    IPLT    IPRT   NSTAN
                   288       0       5       0      0       0       0       0       4       0
                                        JOPER     NWT   LROPT   TRACE
                                            5       0       0       0

                                   MULTI-PLAN ANALYSES TO BE PERFORMED
                                     NPLAN= 1 NRTIO= 1 LRTIO= 1
                 RTIOS=   0.50

      **********      **********      **********      **********      **********

                                   SUB-AREA RUNOFF COMPUTATION

                    SUB-AREA 1 RUNOFF COMPUTATION
                              ISTAQ    ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                               SA-1        0       0       0       0       0       1       0       0

                                          HYDROGRAPH DATA
                   IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                       1       1    0.13    0.00   10.00    0.00   0.000       0       1       0

                                            PRECIP DATA
                            SPFE     PMS      R6     R12     R24     R48     R72     R96
                            0.00   17.50  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                             LOSS DATA
              LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
                  0    0.00    0.00    1.00    0.00    0.00    1.00    1.00    0.10    0.00    0.00

                                      UNIT HYDROGRAPH DATA
                                  TP=  0.40    CP=0.63    NTA=  0

                                          RECESSION DATA
                           STRTQ=   -4.00    QRCSN=    0.00    RTIOR= 1.00

                    UNIT HYDROGRAPH 27 END-OF-PERIOD ORDINATES, LAG=   0.40 HOURS, CP= 0.63   VOL= 1.00
              12.      42.      81.     116.     133.     127.     105.      83.      66.      53.
              42.      33.      26.      21.      17.      13.      11.       8.       7.       5.
               4.       3.       3.       2.       2.       1.       1.

                                      END-OF-PERIOD FLOW
     0
 MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q    MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q
                                                                  SUM  18.48  15.86   2.62   16341.
                                                                     ( 469.)( 403.)(  67.)( 462.73)
```

```
**********        **********        **********        **********        **********

                         SUB-AREA RUNOFF COMPUTATION

SUB-AREA 2 RUNOFF COMPUTATION
              ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE  IAUTO
              SA-2        0       0       0      0      0       1       0      0

                               HYDROGRAPH DATA
        IHYDG   IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
            1     -1    0.02    0.00   10.00    0.00   0.000       0       1       0

                                PRECIP DATA
               SPFE     PMS      R6     R12     R24     R48     R72     R96
               0.00   17.50  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                 LOSS DATA
    LROPT  STRKR  DLTKR  RTIOL  ERAIN  STRKS  RTIOK  STRTL  CNSTL  ALSMX  RTIMP
        0   0.00   0.00   1.00   0.00   0.00   1.00   0.00   0.00   0.00   0.00

                               RECESSION DATA
                  STRTQ=   14.00    GRCSN=    0.00    RTIOR= 1.00

         0                     END-OF-PERIOD FLOW
MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q    MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q

                                                             SUM  18.48  18.48   0.00    7304.
                                                                ( 469.)( 469.)(   0.)( 206.83)


**********        **********        **********        **********        **********

                             COMBINE HYDROGRAPHS

COMBINING HYDROGRAPHS 1,2
              ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE  IAUTO
              SA-2C       2       0       0      0      0       1       0      0


**********        **********        **********        **********        **********

                             HYDROGRAPH ROUTING

ROUTING FLOW THROUGH RESERVOIR
              ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE  IAUTO
                RES       1       0       0      0      0       1       0      0
                                ROUTING DATA
          QLOSS   CLOSS     AVG    IRES   ISAME   IOPT   IPMP          LSTR
            0.0   0.000    0.00       1       1      0      0             0

              NSTPS   NSTDL     LAG   AMSKK       X     TSK    STORA   ISPRAT
                  1       0       0   0.000   0.000   0.000    -381.       -1

STAGE      381.00    382.00    383.00    384.00    384.50    385.00    385.50    386.00    387.00    388.00
           389.00

FLOW         0.00     71.00    196.00    342.00    434.00    546.00    604.00    660.00    759.00    836.00
           836.00

SURFACE AREA=      0.       2.       8.      15.      17.      21.

CAPACITY=          0.       6.      56.     170.     233.     517.
```

ELEVATION=	355.	361.	371.	381.	385.	400.		

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
381.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
385.0	3.1	1.5	597.

DAM BREACH DATA

BRWID	Z	ELBM	TFAIL	WSEL	FAILEL
50.	0.50	355.00	0.50	381.00	382.78

BEGIN DAM FAILURE AT 16.42 HOURS

PEAK OUTFLOW IS 7638. AT TIME 16.74 HOURS

THE DAM BREACH HYDROGRAPH WAS DEVELOPED USING A TIME INTERVAL OF 0.010 HOURS DURING BREACH FORMATION.
DOWNSTREAM CALCULATIONS WILL USE A TIME INTERVAL OF 0.083 HOURS.
THIS TABLE COMPARES THE HYDROGRAPH FOR DOWNSTREAM CALCULATIONS WITH THE COMPUTED BREACH HYDROGRAPH.
INTERMEDIATE FLOWS ARE INTERPOLATED FROM END-OF-PERIOD VALUES.

TIME (HOURS)	TIME FROM BEGINNING OF BREACH (HOURS)	INTERPOLATED BREACH HYDROGRAPH (CFS)	- COMPUTED BREACH HYDROGRAPH (CFS)	= ERROR (CFS)	ACCUMULATED ERROR (CFS)	ACCUMULATED ERROR (AC-FT)
16.417	0.000	169.	169.	0.	0.	0.
16.427	0.010	319.	169.	151.	151.	0.
16.437	0.021	470.	169.	301.	452.	0.
16.448	0.031	621.	169.	452.	904.	1.
16.458	0.042	772.	211.	561.	1464.	1.
16.469	0.052	923.	407.	516.	1980.	2.
16.479	0.062	1073.	683.	391.	2371.	2.
16.490	0.073	1224.	1010.	214.	2584.	2.
16.500	0.083	1375.	1375.	0.	2584.	2.
16.510	0.094	1782.	1764.	18.	2602.	2.
16.521	0.104	2189.	2170.	19.	2621.	2.
16.531	0.115	2596.	2585.	12.	2632.	2.
16.542	0.125	3003.	3001.	2.	2634.	2.
16.552	0.135	3410.	3416.	-6.	2628.	2.
16.562	0.146	3817.	3831.	-13.	2615.	2.
16.573	0.156	4224.	4237.	-13.	2602.	2.
16.583	0.167	4631.	4631.	0.	2602.	2.
16.594	0.177	4944.	5009.	-64.	2537.	2.
16.604	0.187	5257.	5379.	-122.	2416.	2.
16.615	0.198	5570.	5736.	-166.	2249.	2.
16.625	0.208	5883.	6070.	-187.	2062.	2.
16.635	0.219	6196.	6378.	-182.	1880.	2.
16.646	0.229	6509.	6659.	-150.	1730.	1.
16.656	0.240	6821.	6911.	-90.	1640.	1.
16.667	0.250	7134.	7134.	0.	1640.	1.
16.677	0.260	7220.	7327.	-106.	1533.	1.
16.687	0.271	7305.	7490.	-185.	1348.	1.
16.698	0.281	7390.	7621.	-231.	1117.	1.
16.708	0.292	7476.	7722.	-246.	870.	1.
16.719	0.302	7561.	7791.	-231.	640.	1.
16.729	0.312	7646.	7830.	-184.	456.	0.
16.740	0.323	7732.	7838.	-107.	349.	0.
16.750	0.333	7817.	7817.	0.	349.	0.
16.760	0.344	7652.	7766.	-114.	235.	0.
16.771	0.354	7488.	7687.	-199.	36.	0.
16.781	0.365	7323.	7580.	-257.	-221.	-0.
16.792	0.375	7158.	7446.	-288.	-509.	-0.
16.802	0.385	6994.	7283.	-289.	-798.	-1.
16.812	0.396	6829.	7071.	-242.	-1040.	-1.
16.823	0.406	6665.	6810.	-145.	-1185.	-1.
16.833	0.417	6500.	6500.	0.	-1185.	-1.
16.844	0.427	5988.	6141.	-153.	-1338.	-1.
16.854	0.437	5475.	5733.	-258.	-1596.	-1.
16.865	0.448	4963.	5279.	-316.	-1912.	-2.
16.875	0.458	4451.	4781.	-331.	-2243.	-2.
16.885	0.469	3938.	4244.	-305.	-2549.	-2.
16.896	0.479	3426.	3671.	-245.	-2794.	-2.
16.906	0.490	2914.	3069.	-154.	-2950.	-2.
16.917	0.500	2401.	2401.	0.	-2950.	-3.

OVR

STA 8+00

NORMAL DEPTH CHANNEL ROUTING

```
  GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH      SEL
 0.0400   0.0300   0.0400    348.0    365.0     800. 0.00900

 CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
     0.00  370.00   30.00  360.00   60.00  350.00   95.00  348.00  105.00  348.00
   120.00  355.00  230.00  360.00  255.00  365.00
```

STA 13+00

NORMAL DEPTH CHANNEL ROUTING

```
  GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH      SEL
 0.0400   0.0300   0.0400    342.0    353.0     500. 0.01200

 CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
     0.00  360.00   50.00  350.00  144.00  344.00  145.00  342.00  155.00  342.00
   156.00  344.00  300.00  350.00  400.00  353.00
```

STA 20+00

NORMAL DEPTH CHANNEL ROUTING

```
  GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH      SEL
 0.0400   0.0500   0.0400    338.0    350.0     700. 0.00600

 CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
     0.00  350.00  100.00  340.00  354.00  339.00  355.00  338.00  365.00  338.00
   366.00  339.00  660.00  340.00 1000.00  353.00
```

STA 25+00

NORMAL DEPTH CHANNEL ROUTING

```
  GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH      SEL
 0.0400   0.0300   0.0400    336.0    350.0     500. 0.00400

 CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
     0.00  350.00   50.00  340.00  464.00  337.00  485.00  336.00  495.00  336.00
   496.00  337.00 1060.00  342.00 1250.00  350.00
```

STA 30+00

NORMAL DEPTH CHANNEL ROUTING

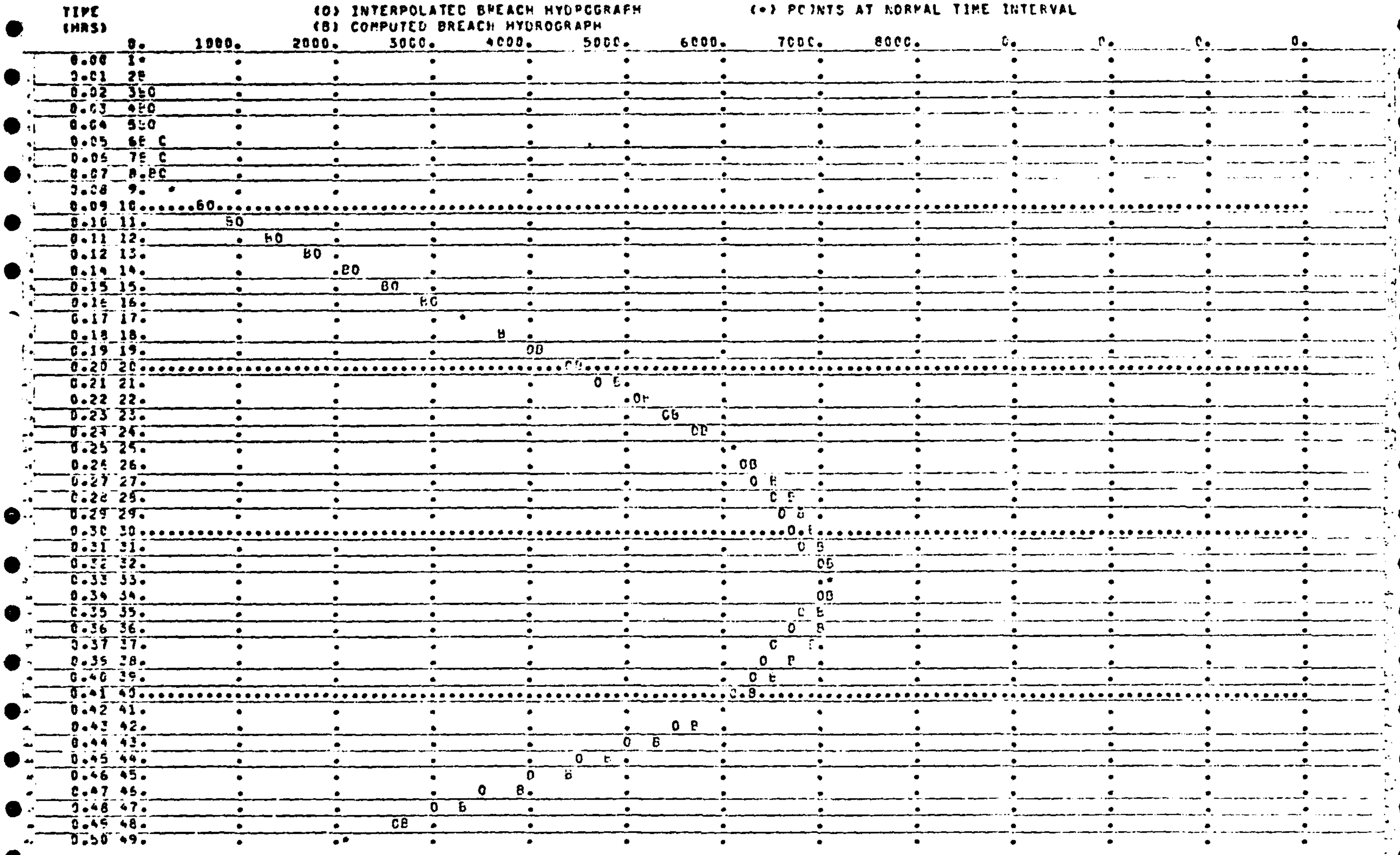
```
  GN(1)    GN(2)    GN(3)    ELNVT    ELMAX    RLNTH      SEL
 0.0400   0.0300   0.0400    333.0    342.0     500. 0.00600

 CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
 [illegible]
```

OVF

STATION RES

(O) INTERPOLATED BREACH HYDROGRAPH
(B) COMPUTED BREACH HYDROGRAPH

(*) POINTS AT NORMAL TIME INTERVAL



THE DAM BREACH HYDROGRAPH WAS DEVELOPED USING A TIME INTERVAL OF 0.010 HOURS DURING BREACH FORMATION.
DOWNSTREAM CALCULATIONS WILL USE A TIME INTERVAL OF 0.083 HOURS.
THIS TABLE COMPARES THE HYDROGRAPH FOR DOWNSTREAM CALCULATIONS WITH THE COMPUTED BREACH HYDROGRAPH.
INTERMEDIATE FLOWS ARE INTERPOLATED FROM END-OF-PERIOD VALUES.

TIME (HOURS)	TIME FROM BEGINNING OF BREACH (HOURS)	INTERPOLATED BREACH HYDROGRAPH (CFS)	-	COMPUTED BREACH HYDROGRAPH (CFS)	=	ERROR (CFS)	ACCUMULATED ERROR (CFS)	ACCUMULATED ERROR (AC-FT)
0.000	0.000	0.		0.		0.	0.	0.
0.010	0.010	34.		0.		34.	34.	0.
0.021	0.021	68.		0.		68.	103.	0.
0.031	0.031	103.		0.		103.	205.	0.
0.042	0.042	137.		0.		137.	342.	0.
0.052	0.052	171.		0.		171.	513.	0.
0.062	0.062	205.		0.		205.	718.	1.
0.073	0.073	239.		63.		176.	894.	1.
0.083	0.083	274.		274.		0.	894.	1.
0.094	0.094	656.		559.		97.	991.	1.
0.104	0.104	1039.		895.		143.	1134.	1.
0.115	0.115	1421.		1267.		154.	1288.	1.
0.125	0.125	1804.		1663.		140.	1428.	1.
0.135	0.135	2186.		2075.		111.	1539.	1.
0.146	0.146	2569.		2495.		74.	1613.	1.
0.156	0.156	2951.		2916.		35.	1648.	1.
0.167	0.167	3334.		3334.		-0.	1648.	1.
0.177	0.177	3677.		3742.		-65.	1583.	1.
0.187	0.187	4021.		4138.		-118.	1465.	1.
0.198	0.198	4364.		4518.		-154.	1311.	1.
0.208	0.208	4708.		4879.		-171.	1140.	1.
0.219	0.219	5051.		5217.		-166.	973.	1.
0.229	0.229	5394.		5532.		-137.	836.	1.
0.240	0.240	5738.		5820.		-82.	754.	1.
0.250	0.250	6081.		6081.		0.	754.	1.
0.260	0.260	6207.		6314.		-107.	647.	1.
0.271	0.271	6333.		6517.		-184.	463.	0.
0.281	0.281	6459.		6689.		-230.	233.	0.
0.292	0.292	6584.		6831.		-246.	-13.	-0.
0.302	0.302	6710.		6941.		-231.	-244.	-0.
0.312	0.312	6836.		7021.		-185.	-429.	-0.
0.323	0.323	6962.		7069.		-108.	-537.	-0.
0.333	0.333	7087.		7087.		0.	-537.	-0.
0.344	0.344	6952.		7075.		-123.	-660.	-1.
0.354	0.354	6816.		7033.		-217.	-877.	-1.
0.365	0.365	6681.		6963.		-282.	-1160.	-1.
0.375	0.375	6546.		6864.		-319.	-1478.	-1.
0.385	0.385	6410.		6723.		-313.	-1791.	-2.
0.396	0.396	6275.		6532.		-258.	-2049.	-2.
0.406	0.406	6139.		6293.		-153.	-2202.	-2.
0.417	0.417	6004.		6004.		0.	-2202.	-2.
0.427	0.427	5511.		5666.		-156.	-2358.	-2.
0.437	0.437	5017.		5280.		-263.	-2621.	-2.
0.448	0.448	4524.		4848.		-325.	-2945.	-2.
0.458	0.458	4031.		4372.		-342.	-3287.	-3.
0.469	0.469	3537.		3857.		-320.	-3607.	-3.
0.479	0.479	3044.		3307.		-263.	-3870.	-3.
0.490	0.490	2550.		2719.		-169.	-4039.	-3.
0.500	0.500	2057.		2057.		0.	-4039.	-3.

```
ELEVATIONS=      355.      361.      371.      381.      385.      400.

                            CREL     SPWID     COQW      EXPW     ELEVL      COQL     CAREA      EXPL
                           381.0       0.0       0.0       0.0       0.0       0.0       0.0       0.0

                                                       DAM DATA
                                          TOPEL     COQD      EXPD    DAMWID
                                          385.0      2.1       1.5      597.

                                                    DAM BREACH DATA
                                   BRWID        Z.     ELBM     TFAIL      WSEL    FAILEL
                                     90.      0.50   355.00      0.50    381.00    381.00

BEGIN DAM FAILURE AT   0.00 HOURS

PEAK OUTFLOW IS     7087. AT TIME    0.33 HOURS
```

```
   **********          **********          **********          **********          **********

                              SUB-AREA RUNOFF COMPUTATION

        SUB-AREA 2 RUNOFF COMPUTATION
                      ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME   ISTAGE   IAUTO
                      SA-2       0        0        0        0       0        1       0        0

                                        HYDROGRAPH DATA
             IHYDG    IUHG    TAREA    SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
               1       -1      0.02     0.00    10.00     0.00    0.000      0        1        0

                                        PRECIP DATA
                   SPFE      PMS      R6       R12      R24      R48      R72      R96
                   0.00     17.50   111.00   123.00   132.00     0.00     0.00     0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                        LOSS DATA
          LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CASTL   ALSMX   RTIMP
            0      0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                                        RECESSION DATA
                      STRTQ=    14.00     GRCSN=    0.00     RTIOR= 1.00

                                        END-OF-PERIOD FLOW
        0
  MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q      MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q
                                                                          SUM  18.48  18.48   0.00     7304.
                                                                             ( 469.)( 469.)(   0.)(  206.83)

   **********          **********          **********          **********          **********

                                        COMBINE HYDROGRAPHS

        COMBINING HYDROGRAPHS 1,2
                      ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME   ISTAGE   IAUTO
                      SA-2C      2        0        0        0       0        1       0        0

   **********          **********          **********          **********          **********

                                        HYDROGRAPH ROUTING

        ROUTING FLOW THROUGH RESERVOIR
                      ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME   ISTAGE   IAUTO
                      RES        1        0        0        0       0        1       0        0
                                        ROUTING DATA
                 QLOSS    CLOSS     AVG     IRES    ISAME    IOPT    IPMP              LSTR
                  0.0     0.000     0.00      1       1        0       0                 0

                      NSTPS    NSTOL     LAG    AMSKK       X      TSK    STORA   ISPRAT
                        1        0        0     0.000    0.000   0.000   -381.      -1

  STAGE      381.00     382.00     383.00     384.00     384.50     385.00     385.50     386.00     387.00     388.00
             389.00

  FLO.         6.00      71.00     196.00     349.00     434.00     540.00     604.00     660.00     755.00     836.00
             836.00

  SURFACE AREA=      0.        2.        8.       15.       17.       21.

  CAPACITY=          0.        6.       56.      170.      233.      517.
```

```
****************************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION        JULY 1978
 LAST MODIFICATION  26 FEB 79
****************************************

RUN  DATE?FRI, MAR 07 1980
     TIME?11:17:54

                    NEC DAM INSPECTION: DACW33-80-C-0012          NED82 DAMBREAK
                    VT. 102, NORTON BROOK DAM, 21-06-79103
                    NO FLOOD
                                           JOB SPECIFICATION
          NO      NHR     NMIN     IDAY      IHR     IMIN    METRC     IPLT     IPRT    NSTAN
         288        0        5        0        0        0        0        0        4        0
                                  JOPER      NWT    LROPT    TRACE
                                      5        0        0        0

                              MULTI-PLAN ANALYSES TO BE PERFORMED
                                 NPLAN= 1 NRTIO= 1 LRTIO= 1
              RTIOS=   0.00

   **********      **********      **********      **********      **********

                                 SUB-AREA RUNOFF COMPUTATION

SUB-AREA 1 RUNOFF COMPUTATION
                  ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                   SA-1       0       0       0       0       0       1       0       0

                                       HYDROGRAPH DATA
      IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
          1       1    0.13    0.00   10.00    0.00   0.000       0       1       0

                                         PRECIP DATA
                  SPFE     PMS      R6     R12     R24     R48     R72     R96
                  0.00   17.50  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                          LOSS DATA
     LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
         0    0.00    0.00    1.00    0.00    0.00    1.00    1.00    0.10    0.00    0.00

                                    UNIT HYDROGRAPH DATA
                          TP=  0.40     CP=0.63     NTA=  0

                                       RECESSION DATA
                     STRTQ=    -4.00    QRCSN=    0.00    RTIOR= 1.00

     UNIT HYDROGRAPH 27 END-OF-PERIOD ORDINATES, LAG=   0.40 HOURS, CP= 0.63   VOL= 1.00
        12.        42.        81.       116.       133.       127.       105.        83.        66.        53.
        42.        33.        26.        21.        17.        13.        11.         8.         7.         5.
         4.         3.         3.         2.         2.         1.         1.

                                     END-OF-PERIOD FLOW
         0
  MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q       MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q

                                                                     SUM   18.48  15.86   2.62  16341.
                                                                         ( 469.)( 403.)(  67.)( 462.73)
```

```
A  NED DAM INSPECTION: DACW33-80-C-0012          NDDD2 DAMBREAK                         PAGE 0002

Y                                       1        1
Y1       1
Y6    .04      .03      .04     336      350      500     .004
Y7      0      350       50     340      484      337      485      336      495      336
Y7    496      337     1060     342     1250      350
K       1   30+00                                   5        1
K1 CHANNEL ROUTING STA 30+00
Y                                       1        1
Y1       1
Y6    .04      .03      .04     333      342      500     .006
Y7      0      342      100     340     1390      334     1395      333     1405      333
Y7   1410      334     1250     340     2200      350
K       1   37+00                                   5        1
K1 CHANNEL ROUTING STA 37+00
Y                                       1        1
Y1       1
Y6    .04      .03      .04     331      350      700     .003
Y7      0      350       50     340       94      333       95      331      105      331
Y7    166      333      140     340      280      350
K       1   55+00                                   5        1
K1 CHANNEL ROUTING  STA 55+00
Y                                       1        1
Y1       1
Y6    .04      .03      .04     298      320     1800     .019
Y7      0      320       10     310      180      300      190      298      210      298
Y7    220      300      250     310      310      320
K       1   65+00                                   5        1
K1 CHANNEL ROUTING  STA 65+00
Y                                       1        1
Y1       1
Y5    .04      .03      .04     280      300     1000     .018
Y7      0      300      240     290      270      281      270      280      310      280
Y7    310      281      350     290      440      300
K       1   70+00                                   5        1
K1 CHANNEL ROUTING STA 70+00
Y                                       1        1
Y1       1
Y5    .04      .03      .04     278      290      500     .004
Y7      0      290      170     280      950      279      990      278     1010      278
Y7   1015      279     1120     280     1250      290
K      99
A
A
A
A
A
```

```
A  NED DAM INSPECTION: DACW33-80-C-0012        NBDB2 DAMBREAK
A  VT. 102, NORTON BROOK DAM, 21-06-79103
A  NO FLOOD
B     288       0       5                                                  4
B1      5
J       1       1       1
J1    .00
K       0    SA-1                                          1
K1 SUB-AREA 1 RUNOFF COMPUTATION                                           1
M       1       1    .132              10
P            17.5     111     123     132
T                                                        1.0     .10
W      .4    .625
X     -4.               1
K       0    SA-2                                          1
K1 SUB-AREA 2 RUNOFF COMPUTATION                                           1
M       1      -1    .023              10
P            17.5     111     123     132
T                                                          0       0
U       1
U1    178
X    14.0       0       1
K       2    SA-2C                                         1
K1 COMBINING HYDROGRAPHS 1,2
K       1    RES                                           1
K1 ROUTING FLOW THROUGH RESERVOIR
Y                                1       1
Y1      1                                                -381      -1
Y4    381     382     383     384   384.5     385   385.5     386     387     388
Y4    389
Y5      0      71     196     349     434     540     604     660     759     836
Y5    836
SA    0.2     2.2     8.4    14.7    17.0    20.9
SE    355     361     371     381     385     400
S+    381
SD    385   3.087     1.5     597
SB     90      .5     355      .5     381     381
K       1    8+00                                  5       1
K1 CHANNEL ROUTING STA 8+00
Y                                1       1
Y1      1
Y6    .04     .03     .04     348     365     800    .009
Y7      0     370      30     360      80     350      95     348     105     348
Y7    180     350     230     360     255     365
K       1   13+00                                  5       1
K1 CHANNEL ROUTING STA 13+00
Y                                1       1
Y1      1
Y6    .04     .03     .04     342     353     500    .012
Y7      0     363      50     350     144     344     145     342     155     342
Y7    156     344     360     350     400     353
K       1   20+00                                  5       1
K1 CHANNEL ROUTING STA 20+00
Y                                1       1
Y1      1
Y6    .04     .03     .04     338     350     700    .006
Y7      0     350     100     340     354     339     355     338     365     339
Y7    366     339     660     340    1000     350
K       1   25+00                                  5       1
K1 CHANNEL ROUTING STA 25+00
```

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7408.	286.7	16.92

PLAN 1 STATION 70+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7254.	281.4	16.92

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	381.00	381.00	385.00
STORAGE	170.	170.	233.
OUTFLOW	0.	0.	540.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.50	382.78	0.00	197.	7838.	0.00	16.74	16.42

PLAN 1 STATION 8+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7775.	364.1	16.75

PLAN 1 STATION 13+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7775.	350.0	16.75

PLAN 1 STATION 20+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7637.	342.0	16.75

PLAN 1 STATION 25+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7609.	340.8	16.83

PLAN 1 STATION 30+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7681.	337.3	16.83

PLAN 1 STATION 37+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7360.	344.7	16.83

PLAN 1 STATION 55+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.50	7729.	304.5	16.9[illegible]

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 0.50
HYDROGRAPH AT	SA-1	0.13	1	302.
	(	0.34)	(	8.54)(
HYDROGRAPH AT	SA-2	0.02	1	154.
	(	0.06)	(	4.37)(
2 COMBINED	SA-2C	0.15	1	330.
	(	0.40)	(	9.33)(
ROUTED TO	RES	0.15	1	7817.
	(	0.40)	(	221.35)(
ROUTED TO	8+00	0.15	1	7775.
	(	0.40)	(	220.15)(
ROUTED TO	13+00	0.15	1	7775.
	(	0.40)	(	220.15)(
ROUTED TO	20+00	0.15	1	7637.
	(	0.40)	(	216.24)(
ROUTED TO	25+00	0.15	1	7609.
	(	0.40)	(	215.47)(
ROUTED	30+00	0.15	1	7681.
	(	0.40)	(	217.51)(
ROUTED TO	37+00	0.15	1	7360.
	(	0.40)	(	208.41)(
ROUTED TO	55+00	0.15	1	7329.
	(	0.40)	(	207.53)(
ROUTED TO	65+00	0.15	1	7408.
	(	0.40)	(	209.77)(
ROUTED TO	70+00	0.15	1	7254.
	(	0.40)	(	205.40)(

STA 37+00

NORMAL DEPTH CHANNEL ROUTING

GN(1)	GN(2)	GN(3)	ELNVT	ELMAX	RLNTH	SEL
0.0400	0.0300	0.0400	331.0	350.0	700.	0.00300

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

0.00	350.00	50.00	340.00	94.00	333.00	95.00	331.00	105.00	331.00
106.00	333.00	140.00	340.00	260.00	350.00				

STA 55+00

NORMAL DEPTH CHANNEL ROUTING

GN(1)	GN(2)	GN(3)	ELNVT	ELMAX	RLNTH	SEL
0.0400	0.0300	0.0400	298.0	320.0	1600.	0.01900

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

0.00	320.00	10.00	310.00	180.00	300.00	190.00	298.00	210.00	298.00
220.00	300.00	250.00	310.00	310.00	320.00				

STA 65+00

NORMAL DEPTH CHANNEL ROUTING

GN(1)	GN(2)	GN(3)	ELNVT	ELMAX	RLNTH	SEL
0.0400	0.0300	0.0400	280.0	300.0	1000.	0.01800

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

0.00	300.00	240.00	290.00	270.00	281.00	270.00	280.00	310.00	280.00
310.00	281.00	350.00	290.00	440.00	300.00				

STA 70+00

NORMAL DEPTH CHANNEL ROUTING

GN(1)	GN(2)	GN(3)	ELNVT	ELMAX	RLNTH	SEL
0.0400	0.0300	0.0400	278.0	290.0	500.	0.00400

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

0.00	290.00	170.00	280.00	950.00	279.00	990.00	278.00	1010.00	278.00
1015.00	279.00	1120.00	280.00	1250.00	290.00				

********** ********** ********** ********** **********

OVN

STA 8+00

NORMAL DEPTH CHANNEL ROUTING

```
 QN(1)   QN(2)   QN(3)   ELNVT   ELMAX   RLNTH     SEL
0.0400  0.0300  0.0400   348.0   365.0    800.  0.00500

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
      0.00  375.00   30.00  360.00   80.00  350.00   95.00  345.00  105.00  348.00
    121.00  350.00  220.00  360.00  255.00  365.00
```

STA 13+00

NORMAL DEPTH CHANNEL ROUTING

```
 QN(1)   QN(2)   QN(3)   ELNVT   ELMAX   RLNTH     SEL
0.0400  0.0300  0.0400   342.0   353.0    500.  0.01200

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
      0.00  360.00   50.00  350.00  144.00  344.00  145.00  342.00  155.00  342.00
    156.00  344.00  300.00  350.00  400.00  353.00
```

STA 20+00

NORMAL DEPTH CHANNEL ROUTING

```
 QN(1)   QN(2)   QN(3)   ELNVT   ELMAX   RLNTH     SEL
0.0400  0.0300  0.0400   339.0   350.0    700.  0.00600

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
      0.00  350.00  100.00  340.00  354.00  339.00  355.00  338.00  360.00  338.00
    365.00  339.00  660.00  340.00 1000.00  350.00
```

STA 25+00

NORMAL DEPTH CHANNEL ROUTING

```
 QN(1)   QN(2)   QN(3)   ELNVT   ELMAX   RLNTH     SEL
0.0400  0.0300  0.0400   336.0   350.0    500.  0.00400

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
      0.00  350.00   50.00  340.00  484.00  337.00  485.00  336.00  495.00  336.00
    496.00  337.00 1060.00  342.00 1250.00  350.00
```

STA 30+00

NORMAL DEPTH CHANNEL ROUTING

```
 QN(1)   QN(2)   QN(3)   ELNVT   ELMAX   RLNTH     SEL
0.0400  0.0300  0.0400   333.0   342.0    500.  0.00500
```

DVF

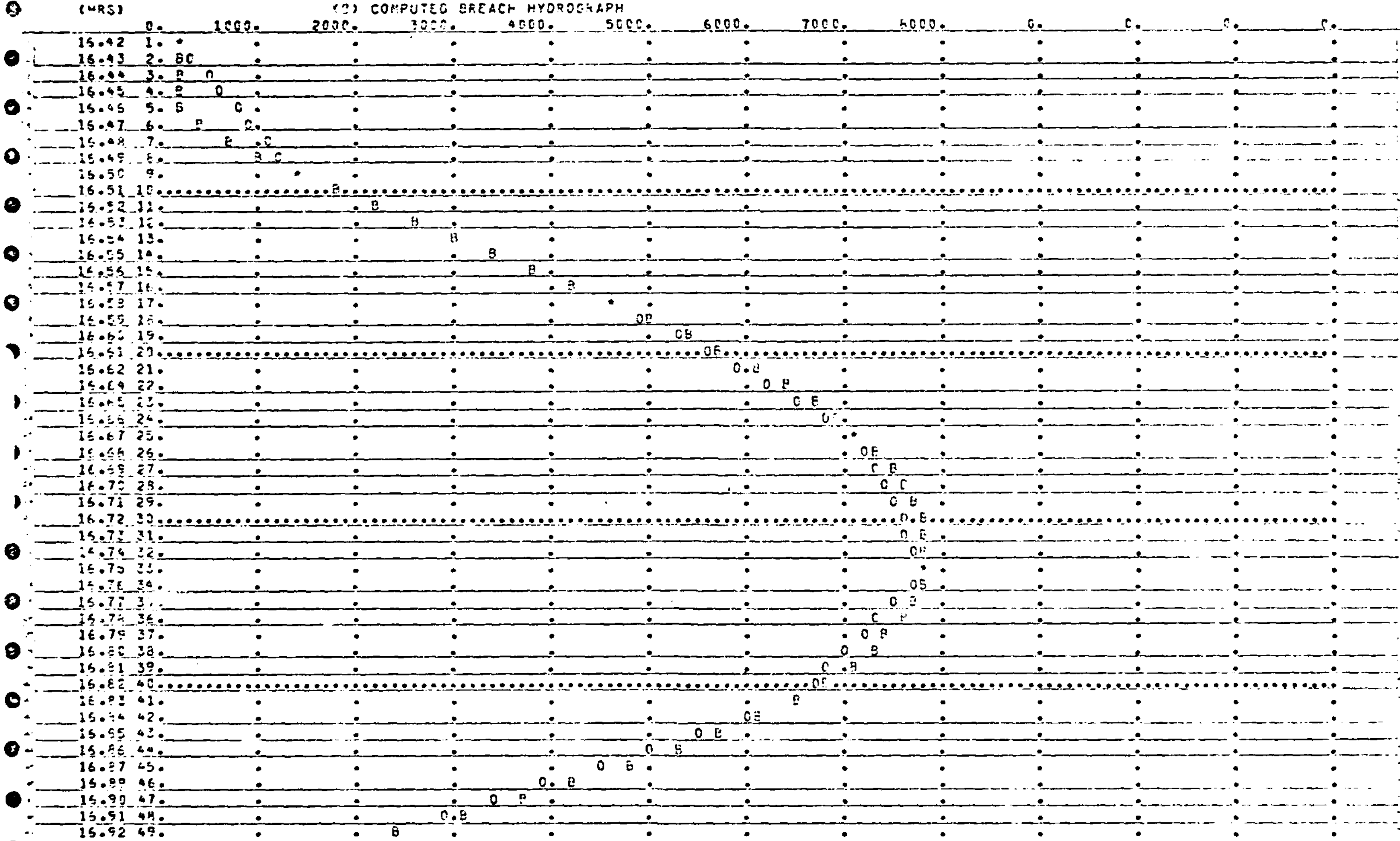
STATION RES

TIME (HRS)

(O) INTERPOLATED BREACH HYDROGRAPH

(B) COMPUTED BREACH HYDROGRAPH

(*) POINTS AT NORMAL TIME INTERVAL



```
NORMAL DEPTH CHANNEL ROUTING          STA 37+00
----------------------------

      QN(1)    QN(2)    QN(3)    ELNVT    ELMAX    RLNTH      SEL
     0.0400   0.0300   0.0400    331.0    350.0     700.  0.00300

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  350.00    50.00  340.00    94.00  333.00    95.00  331.00  105.00  331.00
         106.00  337.00   140.00  340.00   280.00  350.00

NORMAL DEPTH CHANNEL ROUTING          STA 55+00
----------------------------

      QN(1)    QN(2)    QN(3)    ELNVT    ELMAX    RLNTH      SEL
     0.0400   0.0300   0.0400    298.0    320.0    1800.  0.01700

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  320.00    10.00  310.00   180.00  300.00   190.00  296.00  210.00  298.00
         220.00  300.00   250.00  310.00   310.00  320.00

NORMAL DEPTH CHANNEL ROUTING          STA 65+00
----------------------------

      QN(1)    QN(2)    QN(3)    ELNVT    ELMAX    RLNTH      SEL
     0.0400   0.0300   0.0400    280.0    300.0    1000.  0.01600

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  300.00   240.00  290.00   270.00  281.00   270.00  280.00  310.00  280.00
         310.00  281.00   350.00  290.00   440.00  300.00

NORMAL DEPTH CHANNEL ROUTING          STA 70+00
----------------------------

      QN(1)    QN(2)    QN(3)    ELNVT    ELMAX    RLNTH      SEL
     0.0400   0.0300   0.0400    278.0    290.0     500.  0.00400

      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
           0.00  290.00   170.00  280.00   950.00  279.00   990.00  278.00 1010.00  278.00
        1015.00  279.00  1120.00  280.00  1250.00  290.00

          **********          **********          **********          **********          **********
```

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

				RATIOS APPLIED TO FLOWS
OPERATION	STATION	AREA	PLAN	RATIO 1 0.00
HYDROGRAPH AT	SA-1	0.13	1	0.
		(0.34)		(0.00)(
HYDROGRAPH AT	SA-2	0.02	1	0.
		(0.06)		(0.00)(
2 COMBINED	SA-2C	0.15	1	0.
		(0.40)		(0.00)(
ROUTED TO	RES	0.15	1	7087.
		(0.40)		(200.69)(
ROUTED TO	8+00	0.15	1	6959.
		(0.40)		(197.05)(
ROUTED TO	13+00	0.15	1	7048.
		(0.40)		(199.58)(
ROUTED TO	20+00	0.15	1	6741.
		(0.40)		(190.88)(
ROUTED TO	25+00	0.15	1	7032.
		(0.40)		(199.13)(
ROUTED TO	30+00	0.15	1	6710.
		(0.40)		(190.02)(
ROUTED TO	37+00	0.15	1	6511.
		(0.40)		(184.37)(
ROUTED TO	55+00	0.15	1	6551.
		(0.40)		(185.51)(
ROUTED TO	65+00	0.15	1	6478.
		(0.40)		(183.44)(
ROUTED TO	70+00	0.15	1	6053.
		(0.40)		(171.41)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	381.00	381.00	385.00
STORAGE	170.	170.	233.
OUTFLOW	0.	0.	540.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.00	381.00	0.00	170.	7067.	0.00	0.33	0.00

PLAN 1 STATION 8+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6559.	353.8	0.33

PLAN 1 STATION 13+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	7048.	349.7	0.33

PLAN 1 STATION 20+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6741.	341.8	0.42

PLAN 1 STATION 25+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	7132.	340.7	0.42

PLAN 1 STATION 30+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6710.	337.2	0.42

PLAN 1 STATION 37+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6511.	344.1	0.50

PLAN 1 STATION 55+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6551.	304.2	0.50

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6478.	286.2	0.50

PLAN 1 STATION 70+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
0.00	6093.	281.2	0.56

APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

APPENDIX F

REFERENCES

REFERENCES

This is a general list of references pertinent to dam safety investigations. Not all references listed have necessarily been used in this specific report.

1. "Recommended Guidelines For Safety Inspection of Dams, Appendix D", Dept. of the Army, Office of the Chief of Engineers, Washington, D.C., November 1976.

2. "HEC-1 Flood Hydrograph Package, Users Manual", The Hydrologic Engineering Center, U.S. Army Corps of Engineers, January 1973.

3. "Flood Hydrograph Package (HEC-1), Users Manual for Dam Safety Investigations", The Hydrologic Engineering Center, U.S. Army Corps of Engineers, September 1978.

4. HMR 33, "Seasonal Variations of Probable Maximum Precipitation, East of the 105th Meridian for Areas 10 to 1000 Square Miles and Durations from 6 to 48 Hours," U.S. Department of Commerce, NOAA, National Weather Service, 1956.

5. HMR 51, "All-Season Probable Maximum Precipitation, U.S. East of 105th Meridian for Areas from 1000 to 20,000 Square Miles and Durations from 6 to 72 Hours", U.S. Department of Commerce, NOAA, National Weather Service, 1974.

6. HYDRO-35, "Five-to-60 Minute Precipitation Frequency for the Eastern and Central United States", U.S. Department of Commerce, NOAA, National Weather Service, June 1977.

7. "Technical Paper No. 40, Rainfall Frequency Atlas of the United States", U.S. Department of Commerce, Weather Bureau, 1961.

8. Design of Small Dams, United States Department of the Interior, Bureau of Reclamation, Second Edition, 1973.

9. King, Horace W. and Brater, Ernest F., Handbook of Hydraulics, fifth edition, McGraw-Hill Book Co., Inc., New York, 1963.

10. "Flood Hydrograph Analyses and Computations", EM 1110-2-1405, U.S. Army Corps of Engineers, 31 August 1959.

11. "Technical Release No. 55, Urban Hydrology for Small Watersheds", U.S. Department of Agriculture, Soil Conservation Service (Engineering Division), January 1975.

12. "Hydraulic Design of Spillways", EM-1110-2-1603, U.S. Army Corps of Engineers, 31 March 1965.

13. "Standard Project Flood Determinations", EM 1110-2-1411, U.S. Army Corps of Engineers, 26 March 1952.

14. "Hydrologic and Hydraulic Assessment", Appendix D of EC 1110-2-188, U.S. Army Corps of Engineers, 30 December 1977.

15. "Reviews of Spillway Adequacy, National Program of Inspection of Non-Federal Dams", ETL 1110-2-234, U.S. Army Corps of Engineers, 10 May 1978.

16. Hammer, Mark J., Water and Waste-Water Technology, John Wiley & Sons, Inc., New York, 1975.

17. "Hydraulic Charts For the Selection of Highway Culverts", Hydraulic Engineering Circular No. 5, U.S. Department of Commerce, Bureau of Public Roads, December 1965.

18. 33 CFR Part 22, Final Rule, "Engineer and Design; National Program For Inspection of Non-Federal Dams", ER 1110-2-106, U.S. Army Corps of Engineers, September 26, 1979.

19. "Water Resources Data For New Hampshire and Vermont - Water Year 1977", USGS Water-Data Report NH-VT-77-1, U.S. Geological Survey, Boston, Ma., 1978.

20. "Climatological Data - May 1979 - New England", Volume 91, No. 5, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina.

21. "Climatological Data - Annual Summary - New England", Volume 90, No. 13, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina.

DATE

ILMED

-8